Drainage Standards

The City of Abilene, Texas

CITY OF ABILENE

Effective May 1, 2007

DRAINAGE STANDARDS

Of

The City of Abilene, Texas

These drainage standards have been developed pursuant to Section 23-264 of the Subdivision Regulations and were adopted by the City Council of the City of Abilene via Resolution Number 45-1985, with an effective date of April 11. 1985. These were approved by John Stocktin, P.E., City Engineer.

On April 25, 2007 these drainage standards were revised by Bob Lindley, P.E., C.F. M., City Engineer and Terry Pribble, P.E., C.F.M., Hydraulics Engineer.

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Drainage Standards
Of the City of Abilene

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*Please note that this example is for the storm events from 5 thru 100 year as per the previous edition of the Drainage Standards of the City of Abilene.

1.0 INTRODUCTION

1.1 Purpose

The purpose of this manual is to establish standards and design criteria necessary for the control of surface drainage in the City of Abilene, Texas. By establishing these standards and design criteria the City of Abilene will be able to obtain effective storm drainage protection for new and existing development.

Standards and criteria as established by this manual are intended to provide unity in analysis and system design. Adherence with this manual will allow identification of the requirements, the analysis of the rainfall, determination of the runoff, methods of collection, and conveyance of these storm waters.

1.2 Scope

The scope of this manual is intended to provide, for the designer, the basic requirements and design principles to properly evaluate conditions for the systematic design of a storm drainage system. These principles provide the basic guidance, but sound engineering judgment must always be applied. Any deviation from the criteria and principles of this manual must be approved by the City Engineer of Abilene, Texas.

1.3 Drainage Systems

Drainage systems for the City of Abilene may be categorized into two systems. Major drainage systems are those intended to convey larger flows and are evaluated with respect to the 100-year frequency storm. A 100 year storm is a storm that has a 1% chance of being exceeded in any one year period. These major systems are considered to provide flood protection. Minor drainage systems consist of street gutters, inlets and pipes and function to provide relief from nuisance type floods, such as those occurring from a 5-year frequency storm. A 5 year storm is a storm that has a 20% chance of occurring every year. A major system will protect us from loss of property and life, and a minor system will provide for convenience and ease of travel.

1.4 Developer Responsibilities

A. Drainage Plan

1. All drainage plans shall be formulated and implemented under the direct supervision of a registered professional engineer, licensed by the State of Texas; plans submitted for final approval shall bear the signature of the submitting engineer along with the following statement:

   > "I hereby certify that I am familiar with the adopted ordinances and regulations of the City of Abilene governing detention and drainage facilities; These plans have been prepared under my supervision; and the foregoing drainage plan complies with the intent and general requirements of the City of Abilene."

2. Drainage Plan Contents. A drainage plan shall consist of engineering drawings, contour maps, and all supporting engineering calculations, as applicable to the land area covered by the plan. This is required to demonstrate full compliance with the requirements of the Abilene Stormwater Management Ordinance and Abilene's adopted Drainage Standards. A plan shall include all pertinent information required by the City Engineer and may include, but is not limited to, any of all of the following elements:

   a. An engineering report dealing with the applicable provisions of the adopted Abilene Drainage Standards, clearly setting forth the scope of the engineering problems and the proposed solutions.

   b. An engineering hydrologic analysis of stormwater runoff under existing site conditions and under proposed developed site conditions in accordance with City Planning Department's Land Use Plans and consultation with the City Engineer.

   c. An engineering hydraulic analysis for the control and conveyance of stormwater runoff under proposed developed conditions.

d. The location of all existing drainage channels, subsurface drainage facilities and other public and private utilities.

e. The on-site 100-year flood boundaries of any major drainage systems.

f. The proposed method of handling all runoff from the development and a demonstrated capability to handle upstream drainage assuming fully developed condition.

g. Proposed fill or other structure elevating techniques, levees, channel modifications, and detention facilities.

h. Detention facility computations comparing inflow and outflow rates to establish maximum storage volume and peak discharge rate requirements and to demonstrate maintenance of pre-development runoff condition.

i. The location and size of all existing and proposed drainage easements and areas.

j. The location, size and character of all temporary and permanent erosion and sedimentation control facilities, with description detailing all on-site erosion control measures which will be established and maintained during all periods of development and construction.

k. The pre-development cross sectional conveyance shall be preserved under all circumstances involving fill in the Floodplain.

l. In addition to the requirements of detentions and pre-development conveyance preservation, the loss of pre-development Floodplain storage provided by a natural creek shall be compensated to the greatest extent possible.

3. To the maximum extent possible, drainage plans shall be fully documented on a topographic map that accurately delineates all existing and proposed drainage facilities such as streets, storm sewers, natural and manmade channels, swales, etc. All existing and proposed floodplain and floodway boundaries and drainage and/or detention easements shall be shown on this map. Where significant drainage area exists outside the specific development tract, a second, larger scale map may be used to delineate contour and the offsite tributary drainage area(s). The exact boundaries of all proposed drainage sub-areas shall be delineated on this map and the pertinent discharge rates from each sub-area shall be listed at the point of discharge. Where physical improvements are proposed to be made as part of this drainage plan implementation, the design characteristics and hydraulic capacities of the proposed facilities and the pertinent hydraulic loadings shall also be delineated on this map.
4. Insure that all dead-end streets have a drainage plan for water flow away from the end of the street. Where the extension of a dead-end street is anticipated the flow away from the point will be provided on the current project.

B. Design Check List For Construction Plans

To assist in the preparation of a complete and standardized set of drainage and construction plans, a check list of data to be included on the plans is presented:

1. Drainage Area Map will be appropriate to the drainage area.
2. Drawing numbers, date, north arrow, signature blocks, match lines.
3. Subdivision name.
4. Names and widths of streets.
5. Easements and rights-of-way.
6. Survey Data: Bench marks for horizontal and vertical control.
7. Street profiles with gutter flow line grades and typical street cross sections.

8. Pipe size, grade, type, class, length, flow required, flow provided, and hydraulic grade line.
9. Manhole size with invert elevation shown.
10. Inlet size with invert elevations shown.
11. Channel size, slope, and plotted water surface of the 100-year flow, cross section, and discharge velocities (See Table 7.1).
12. Location, horizontally and vertically, of all utility lines.
13. Soil boring logs when available.
14. Structural details.
15. Detention facility location, storage volume, principle spillway design and associated appurtenance details.
16. Erosion and sediment control device location and details as necessary.
17. The Engineer will insure that other drainage is not trapped because of his development.

C. Submittal of Computations

A complete set of design calculations on the appropriate calculation sheets shall be submitted and approved by the City Engineer for all drainage related projects.

D. Applicability

These standards will apply to any drainage plan not previously constructed (whether previously approved or a new submittal.)

2.0 STANDARDS

2.1 General

All drainage systems shall be designed in accordance with the requirements of this section and manual. City of Abilene Minimum Acceptable Design Details shall be utilized.

2.2 Major Drainage Systems

A. Natural Streams or Flowage Areas

1. Unimproved drainage ways scheduled to remain in the natural state shall be dedicated to the City of Abilene either by title or easement, and platted to the 100-year flood way limit.
2. Procedure for design discharge flows, erosion protection, and water surface elevations shall be in accordance with Section 7.2.
3. Where the proposed improvement encroaches into a natural stream area the floodway shall be dedicated in similar manner as Section 2.2, Item A.1 to the City of Abilene. The floodway limits shall be determined in accordance with National Flood Insurance standards.

B. Improved Channels

1. Conveyance for at least the 10-year flood shall be conveyed within the channel banks.
2. The 100-year runoff shall be conveyed within the channel right-of-way and/or dedicated easement.
3. Channel right-of-way and/or dedicated easement shall provide for maintenance access.
4. Unlined channels shall have side slopes no greater than 3:1 and bottom widths not less than 8 feet unless prohibited by existing facilities.
5. Lined channels may have side slopes as steep as 2:1 provided no slope maintenance is required.

6. Procedure for design discharge flows, erosion protection, and water surface elevations shall be in accordance with Section 7.

C. <u>Detention Storage</u>

1. Detention storage is used principally to attenuate the increased runoff caused by urbanization.

2. Detention storage areas shall have the capacity and outlet system to reduce flows for the 2-year through 100-year frequency storms to a level not exceeding pre-development rates. Design criteria shall be in accordance with Section 6.

3. Maintenance of detention areas shall normally be the responsibility of the City of Abilene except for certain multi-purpose areas such as parking areas, roof top storage or as determined by the City Engineer. In such cases a filed maintenance agreement in the Public Records of Taylor Co. shall be required.

4. Detention sites receiving City of Abilene maintenance shall be dedicated to the City of Abilene either by title, easement or plat. The dedicated area shall include all land inundated by the 100-year flood plus additional area as necessary to provide for appropriate maintenance and adequate ingress and egress.

5. Discharge from detention storage areas shall not cause downstream erosion as per Section 7.1 and Section 9.5 and 9.6.

6. An emergency spillway shall be provided and sized to convey the excess 100-year flow which is not stored or conveyed by the principal spillway.

7. Detention ponds shall comply with the following site standards unless variance from the City Engineer is obtained:

   a. Concrete paved or rock lined flow line in accordance with Minimum Acceptable Design Details shall be provided in flow line of basin designed to handle flows more frequent than natural rainfall.

b. Concrete pavement is not required in areas where design storage will not receive flows more frequent than natural rainfall.

c. Erosion protection as per Section 3 shall be provided at the inflow and outflow of each structure.

d. Basins which have disturbed areas from the natural state shall be seeded for erosion control as per Soil Conservation Service/Texas Department of Transportation standards. Prior to acceptance the grass shall be fully established, or a financial guarantee for the same shall be deposited with the City.

e. Basin shall be designed for complete drainage resulting in a Dry Pond unless otherwise approved by the City Engineer.

f. Hydrologic routing through the detention pond of discharges from the 2, 5, 10, 25, 50, and 100 year storms shall be performed to ensure that post development runoff is equal to or less than pre-development runoff under a range of storm frequencies. In an effort to standardize computational procedure, the City requests that the Engineer use either HEC1 (U.S. Army Corps of Engineers) or TR20 (National Resource Conservation Commission) Computations using design software other than HEC1 or TR20 may not be accepted by the City Engineer.

8. Where applicable, dam design and safety requirements as set forth by the State of Texas shall be met.

D. Emergency Overflows

1. Emergency overflows or overland swales shall be provided at all mid-block low points or other low points.
2. Overflow routes shall convey within the right-of-way the 100 year storm.
3. A flood limit easement equal to the spread of water from the 100-year storm shall be dedicated to the City of Abilene. The minimum width shall be 15 ft.

4. Overland flow routes shall be seeded for erosion control in accordance with standards of Texas Department of Transportation or other methods acceptable to the City Engineer.

E. Street Rights-of-way

1. All new streets designed to convey storm water runoff shall convey the 100-year flow within the right-of-way limits and/or specifically dedicated easements.

F. Bridges and Culverts

1. Where bridges and/or culverts are installed on major drainage systems, they shall be designed for the 25-year storm event and to meet FEMA standards of zero rise during the 100-year event.
2. The 100-year flow will not be permitted to overflow specified street bridges in accordance with the approved City Plan.
3. Design criteria shall be in accordance with Section 9.0 of this manual.
4. Headwalls, rip rap or other approved erosion protection shall be provided at the upstream and downstream ends of the culvert barrel(s) which conform the Minimum Acceptable Design Details.
5. Culverts length plus headwalls shall conform to the Minimum Acceptable Design Details.

2.3 Minor Drainage Systems

A. Streets

1. Street design and layout should function to provide the initial conveyance system for stormwater runoff for most if not all developments. Street layout should follow existing topography and drainage patterns as closely as practicable. The hydraulic design of a street be such that reasonable access by emergency vehicles and personnel is ensured during rainstorm events up to and including the 100-year storm.
2. All streets designed to convey stormwater runoff shall convey the 100-year flow within the right-of-way limits and/or specifically dedicated easements.

3. For the purposes of maintaining reasonable traffic flow and safety at all times, and controlling nuisance flooding during all storm events up to and including a 5-year storm, all streets that will convey stormwater runoff shall be designed in accordance with the following standards:
   a. Residential and Collector streets – For all flows up to the 5-year storm, flowing water shall not exceed six inches (6") in depth at any point within the traffic lanes, including intersections, with traffic lanes being defined as the central twenty-two feet (22') of pavement.
   b. Arterial Streets – For all flows up to the 5-year storm, one twelve foot (12') wide lane in the central portion of the street shall remain dry.
   c. Divided Arterial Streets and Freeways – For all flows up to the 5-year storm, one twelve foot (12') wide lane in each direction shall remain dry.

[NOTE: The 5-year frequency storm with 6" of water on the street will equate to about 9" of water on the street for a 100-year frequency storm.]

4. The surface of an arterial street shall not be crossed with stormwater runoff from rainfall events at or below the 5-year frequency. Flows from events greater than the 5-year storm may overtop the roadway surface so long as the flow direction continues along the same line as that for lesser storm event flows.
5. All existing naturally occurring channels which carry stormwater at a rate greater than 10 c.f.s. during a 5-year storm event shall be maintained as such through any new development, and the developer shall not be allowed to close such a channel. In the event a channel as just described comes up to a street, the stormwater shall be carried underneath the street in a culvert structure and will not be discharged into the street.

B. Storm Drain Inlets and Pipes

1. When the 5-year frequency flood exceeds the standard street section spread limits, the following could be implemented:

   a. A storm drainage system consisting of inlets, pipes, manholes and associated appurtenances.

   b. Removal of all or part of the storm flow from the street into an alternative conveyance.

   c. Street widening as approved by the City Engineer.

   d. Design and installation of non-standard gutter sections.

NOTE: These are only suggestions and are not meant to be requirements. The Design Engineer may implement other methods if approved by the City Engineer.

2. City of Abilene standards or other standards approved by the City Engineer may approve be utilized for each component part of the drainage system.

3. Design criteria shall be in accordance with the applicable sections of this manual.

4. The storm drain system shall conform to the following criteria unless otherwise approved by the City Engineer:

   a. Minimum velocity with the pipe flowing full shall be 2.5 feet per second. The maximum velocity of storm drain (collectors) shall be limited to 15 feet per second. The maximum velocity of storm drain (mains) shall be limited to 12 feet per second.

   b. Minimum storm drain pipe diameter shall be 18 inches circular pipe or #3 pipe arch.

   c. Pipe diameters shall not decrease downstream, except when outlet control for detention may be applicable.

   d. Pipe crowns at change in sizes should be set at the same elevation.

   e. Vertical curves in the conduit will not be permitted except where siphons are approved.

f. Maximum manhole spacing:

Pipe Size	Maximum Spacing
18 - 36 in.	600 ft.
42 - 60 in.	1,000 ft.
Larger than 60 in.	No limit

Manholes shall be placed at horizontal P.I. alignment changes.

g. Minimum pipe cover over the top of the pipe to be not less than 1.5 ft. unless approved by the City Engineer, based upon manufacturer's recommendations.

h. The calculated hydraulic grade line of a closed drainage system shall not be more than the maximum control elevation over the street surface (Sec. 2.3 A 1 & 2).

i. Short radius bends may be used only on 24" and larger pipes at a junction or bend. A manhole shall always be located at the end of such short radius bends.

C. Roadways without Curb and Gutter

1. Culverts placed in roadside ditches shall pass the 5-year frequency flow, or pass the hydraulic capacity of the ditch without overtopping the road, driveway, etc., more than 6 inches.
2. Headwalls and endwalls, as per Section 9.3, and meeting City of Abilene Minimum Design Details or other approved details shall be provided at the upstream and downstream ends of ditch culverts.
3. Culverts shall have sufficient length to permit a 4:1 slope extending from the shoulder limit to flow line of the pipe where a vertical headwall is not used.
4. Minimum culvert diameter shall be 18 inches or #3 pipe arch.
5. Design criteria shall be in accordance with the applicable sections of this manual.

6. Culverts parallel to roadway shall have sufficient length to permit a 4:1 slope extending from the top back at curb or outside edge of shoulder or nearest roadway edge and 12 inch minimum rip rap around pipe.

D. Roadside Ditches

1. Roadside ditches shall convey the 10-year flow without overflowing the banks (edge of pavement to right-of-way).
2. Maximum ditch velocities shall not exceed those identified in Section 7 without providing erosion protection.

E. Erosion and Sediment Control

1. All improved ditches with erodable materials shall be provided with erosion protection for design velocities exceeding those in Section 7 of this manual.
2. Sediment control shall be provided for all residential developments of two or more acres and all commercial and industrial tracts of one-half acre or more.
3. Sediment shall not be conveyed into improved drainage facilities, public rights-of-way, or onto adjacent property. Recommendations for sediment control are presented in Section 3 of this manual, or as otherwise approved by the City Engineer.
4. Entrapped sediment shall periodically be removed and redistributed within the development site or otherwise properly disposed. Detention basins, until accepted by the City for maintenance, shall be continuously monitored and maintained as necessary to maintain design storage and flow conditions.

3.0 EROSION AND SEDIMENTATION CONTROL PLAN

3.1 General

Land development activities expose disturbed soils to precipitation and surface stormwater runoff. Protective vegetation is reduced or removed; the topography of the land is altered, excavated material is stockpiled, and the physical properties of the soil itself are changed.

These alterations to the land during construction contribute to an increase in the natural erosion and sedimentation process. For this reason, an erosion and sediment control program must be instituted during the construction phase. Prior to the completion of a subdivision and acceptance of public improvements, rights-of-way, and easements each lot shall have a sediment barrier constructed, designed for a 2-year storm (such construction to be covered by a Maintenance Bond) and maintained until the lot is stabilized, or one (1) year after acceptance of the subdivision by the City as per said maintenance bond. An erosion and sediment control program shall be utilized to prevent sedimentation damage to areas and streams below the development site until the site is stabilized. The following practices are recommended for erosion and sedimentation control.

3.2 Sediment Barriers

Sediment barriers intercept runoff and filter off sediment carried by it prior to its discharge into a water course or onto down grade neighboring properties. Properly constructed and maintained, these structures remove the bulk of coarser sediment from runoff leaving a construction site a defectively reduce the velocity and hence erosive capacity of runoff.

Two structures recommended as sediment barriers below a construction site are the straw bale barrier and the silt fence. The straw bale barrier consists of standard rectangular straw bales placed end to end (tied together with nylon binder twine or wire) and adequately staked to the ground. The silt fence consists of a filter fabric attached to a wire mesh fence of suitable height.

These methods are applicable for small drainage areas and either Structure may be used in a minor swale or ditch line with a runoff contributing area less than two acres. Care should be taken in determining the number and spacing of these structures for larger drainage areas.

3.3 Storm Drain Inlet Protection

Storm drain inlets intercepting flow from a disturbed drainage area should be provided adequate protection against sediment entering the storm drain line. Recommended practices for a standard grate drop inlet include a straw bale barrier, a burlap fence filter, and a block and gravel filter.

The straw bale barrier and burlap fence are similar to the sediment barriers described in Section 3.2. These structures are constructed along the outside perimeter of the inlet. Good judgment should be used in determining the adequacy of these filters for larger concentrated flows which may reduce their structural stability.

The block and gravel filter is used where heavy concentrated flows are expected and where an overflow capacity is necessary to prevent excessive ponding around the structure. It consists of cinder blocks placed lengthwise around the perimeter of the inlet and wrapped with wire mesh. A suitable coarse aggregate is then piled against the wire mesh to the top of the block barrier. The wire mesh should have ½ inch openings and the aggregate should be of suitable size and gradation to effectively filter sediment.

Recommended practices for a standard depressed curb inlet include a straw bale, silt fence, and a block and gravel sediment filer. These structures are similar in purpose and design as mentioned for a grate drop inlet. The design and placement of barriers will be as approved by the City Engineer.

All previously mentioned structures should be periodically checked and maintained to provide optimum protection against sediment entering the storm drain line.

3.4 Detention Basin

Small detention basins are an alternative measure to sediment barriers for preventing sediment to be carried onto down land areas. Basins should be sized with proper outlet works to detain stormwater runoff for a length of time suitable for the settling of most sediment carried off with the stormwater. Runoff from a construction site must be effectively diverted to the basin for implementation of this method. Periodic cleaning of the detention basin is necessary to prevent loss of storage volume capacity.

3.5 Erosion Control

Erosion control prevents sediment from inundating areas below a construction site by preventing displacement of soil located within the construction zones due to runoff. Various methods such as the diversion of upland stormwater runoff, temporary seeding and mulching of the site, and surface roughening to reduce runoff velocity and increase infiltration will reduce the erosion of denuded land before stabilization is established.

3.6 For development of land greater than 1.0 acre a stormwater pollution prevention plan prevent plan (SWP3) will be considered as fulfillment of this section.

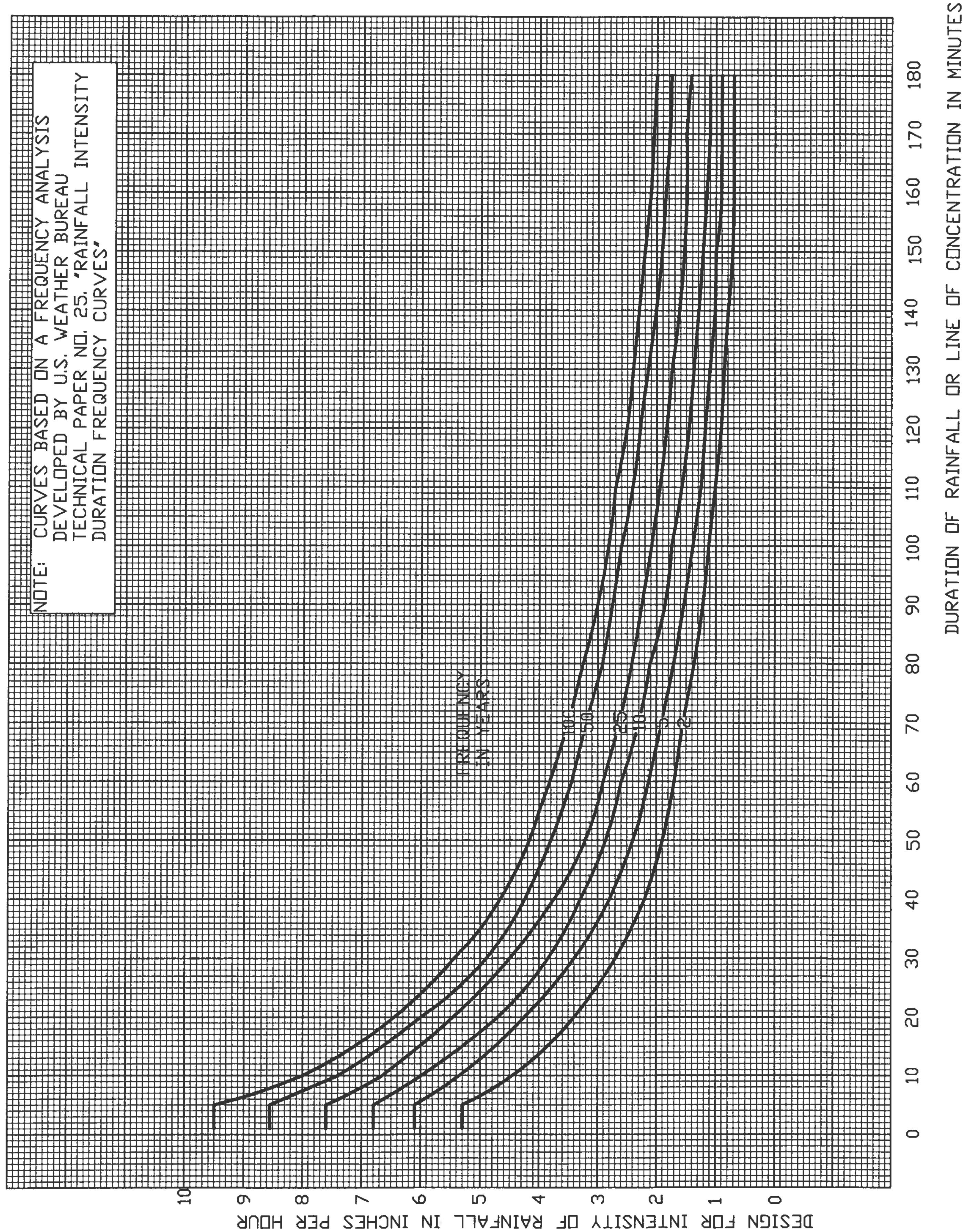
3.7 An erosion plan is required to be approved by the City Engineer for development of land less than 1.0 acre.

4.0 RAINFALL

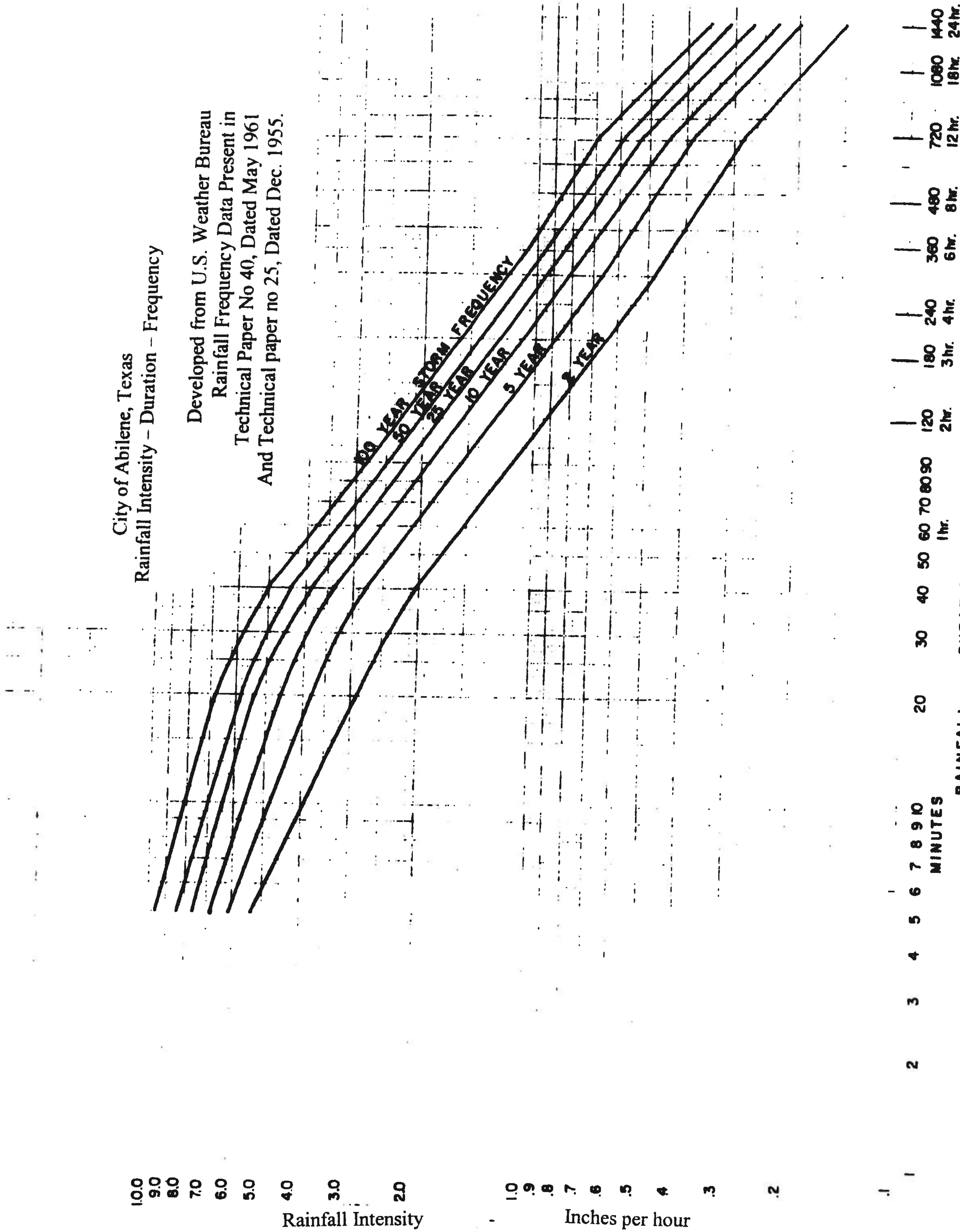
4.1 Abilene Design

Rainfall rates for engineering design purposes shall be estimated in accordance with standard technical information provided by the U.S. Weather Bureau. Design rainfall intensities for the Abilene Area, as a function of duration of rainfall or time of concentration, may be acceptably determined using the relationships depicted by Figures 4.1 and 4.2. The relationships in Figure 4.1 are generally applicable for small watersheds where time of concentration is no more than about three (3) hours.

U.S. Weather Bureau Technical Papers No. 25 and No. 40 are acceptable sources of guidelines and procedures for addressing rainfall on large watersheds or over long durations. The information, procedures and guidelines contained within those publications should be utilized by the Design Engineer when deemed necessary for more accurate representation of a rainfall event over a particular drainage basin. For larger watersheds with time of concentration greater than three (3) hours, adjustments to runoff rates as a percentage of point rainfall intensity may be warranted. In such cases, an acceptable guideline is the area-depth relationship depicted in Figure 4.3.



NOTE: CURVES BASED ON A FREQUENCY ANALYSIS
DEVELOPED BY U.S. WEATHER BUREAU
TECHNICAL PAPER NO. 25. "RAINFALL INTENSITY
DURATION FREQUENCY CURVES"
FREQUENCY
IN YEARS
100
50
25
10
5
2
DESIGN FOR INTENSITY OF RAINFALL IN INCHES PER HOUR
0
1
2
3
4
5
6
7
8
9
10
DURATION OF RAINFALL OR LINE OF CONCENTRATION IN MINUTES
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
170
180


City of Abilene, Texas
Rainfall Intensity – Duration – Frequency
Developed from U.S. Weather Bureau
Rainfall Frequency Data Present in
Technical Paper No 40, Dated May 1961
And Technical paper no 25, Dated Dec. 1955.
100 YEAR STORM FREQUENCY
50 YEAR
25 YEAR
10 YEAR
5 YEAR
2 YEAR
1 2 3 4 5 6 7 8 9 10 MINUTES
20 30 40 50 60 70 80 90 1 hr.
120 2 hr.
180 3 hr.
240 4 hr.
360 6 hr.
480 8 hr.
720 12 hr.
1080 18 hr.
1440 24 hr.
RAINFALL DURATION
10.0 9.0 8.0 7.0 6.0 5.0 4.0 3.0 2.0 1.0 .9 .8 .7 .6 .5 .4 .3 .2 .1
Rainfall Intensity - Inches per hour

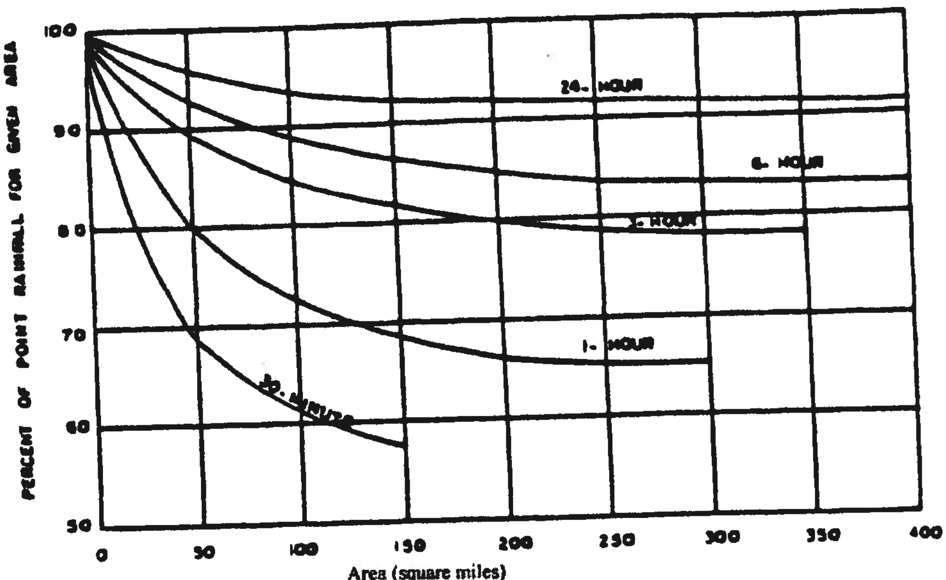


Fig. 4-2 Area-depth curves for use with
Duration – frequency values

From U.S. Dept of Commerce
Technical Paper No. 40

FIGURE 4.3

5.0 RUNOFF

5.1 General

Runoff will be determined using established procedures and criteria.

5.2 Rational Method

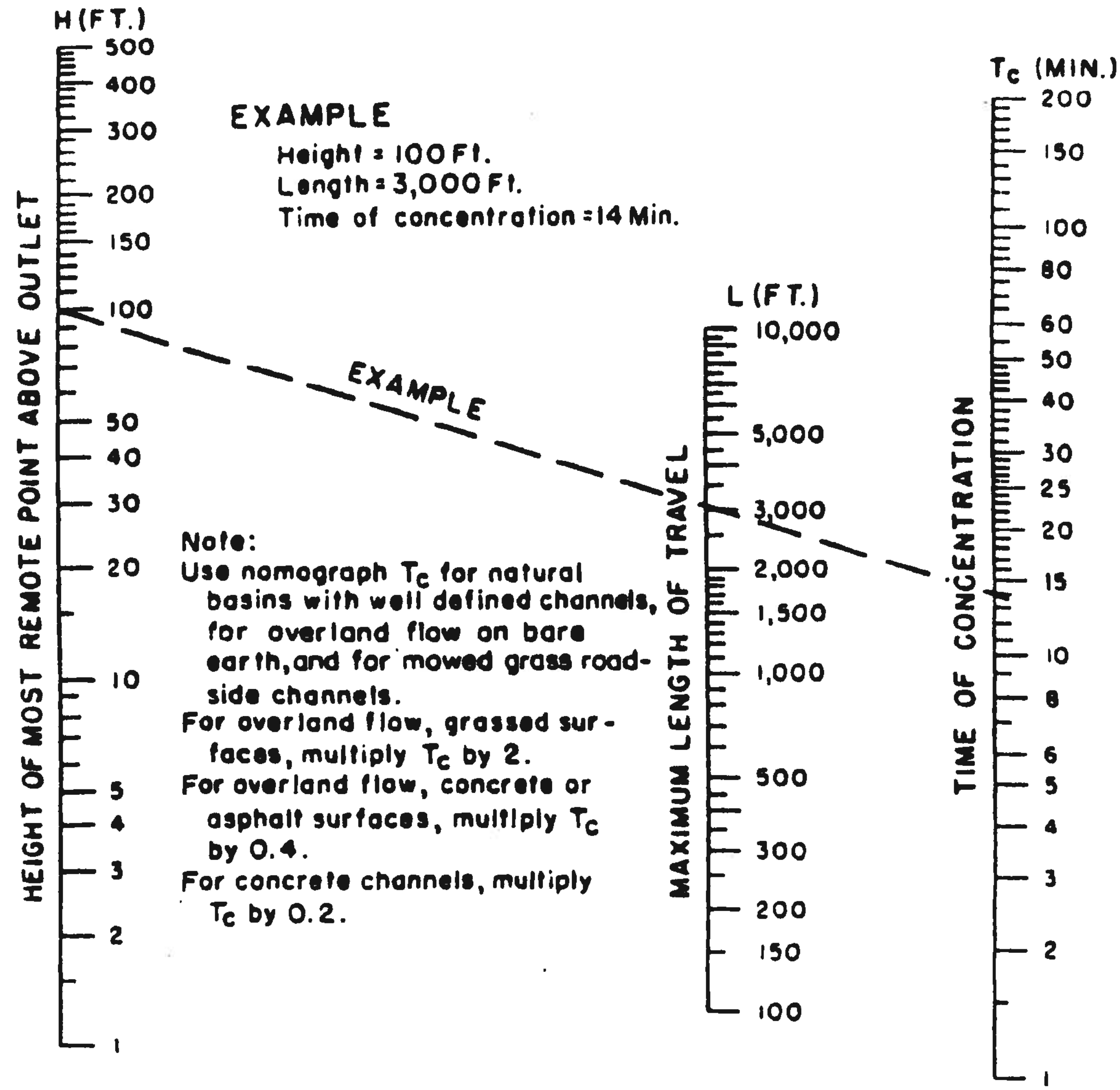
For areas less than about 200 acres, the Rational Method may always be used. When the drainage area being analyzed exceeds 250acres the Design Engineer should carefully establish the applicability of the Rational Method, and its use must have the concurrence of the City Engineer. Careful consideration must be given to each factor involved in the Rational Method for a particular situation in order to insure obtaining a realistic peak runoff value.

A. Runoff Coefficient.

Table 5.1 lists some runoff coefficients adapted from the American Society of Civil Engineers, manuals of Engineering Practice No. 37. These values are applicable to storm frequencies of 2-year to 100-year rate of return. Where land uses other than those listed in Table 5.1 are planned, a coefficient shall be developed utilizing valves comparable to those shown. A written analysis by a Professional Engineer will govern over values shown in Table 5.1.

1. Coefficient Adjustment Factors

Less frequent, higher intensity storms will require the use of higher coefficients in the more pervious land areas because infiltration and other obstructions have a proportionately smaller effect on actual runoff. Runoff coefficient adjustment factors are given in Table 5.2 for respective storm frequencies. Coefficients are multiplied by these factors to obtain corrected coefficients for the respective storm frequencies. In no case shall the coefficient be greater than 1.0.



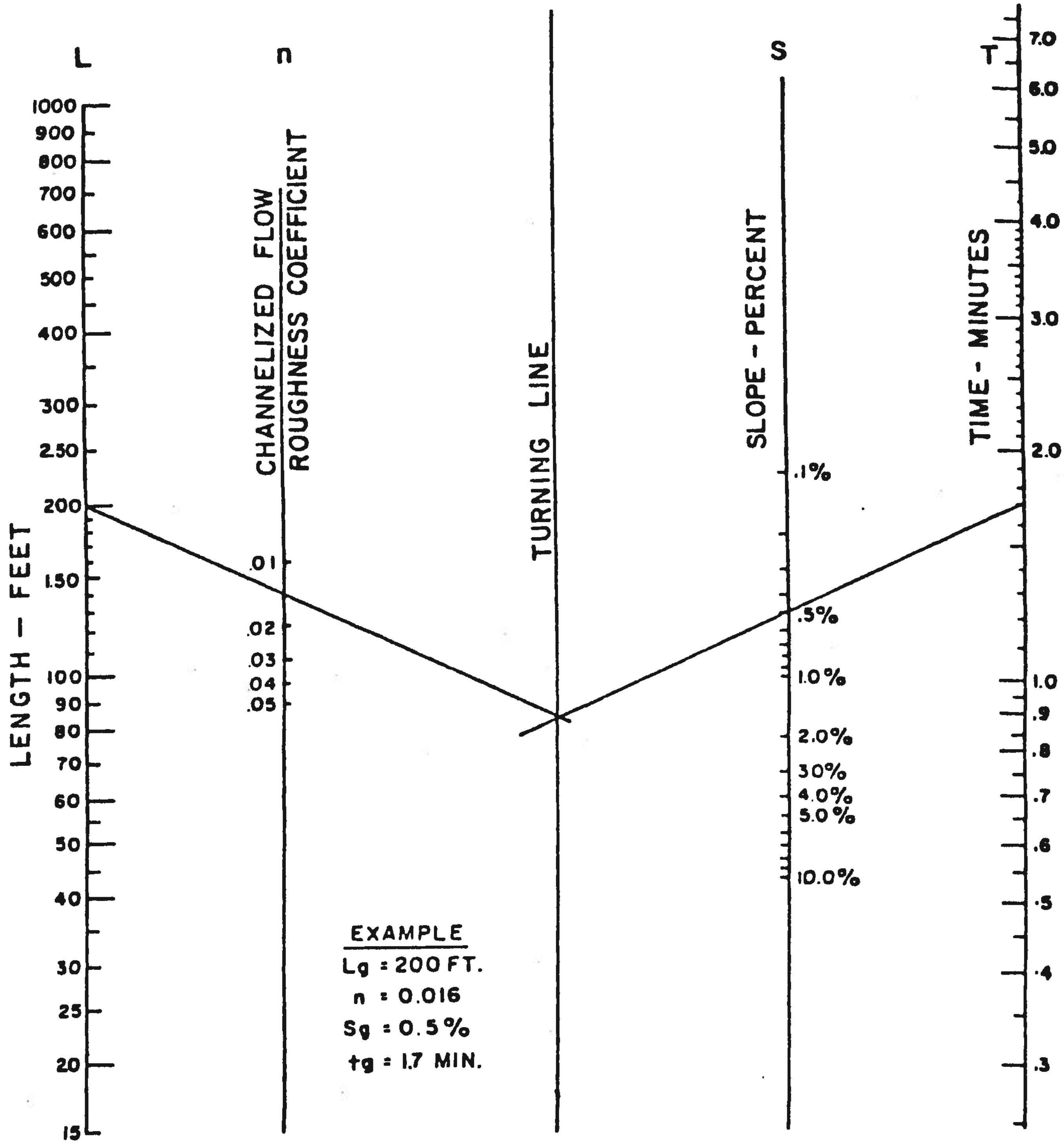
Based on study by P. Z. Kirpich,
Civil Engineering, Vol. 10, No. 6, June 1940, p. 362

TIME OF CONCENTRATION OF SMALL DRAINAGE BASINS

TIME OF CONCENTRATION OF SMALL DRAINAGE BASINS IN MINUTES

$$T = 0.0078 \frac{L^{.77}}{S^{.385}}$$

Figure 5 1



NOMOGRAPH
TIME OF CONCENTRATION
FOR CHANNELIZED FLOW

NOTE: GUTTER FLOW BASED UPON AVERAGE CONDITIONS IN A PARABOLIC STREET SECTION. CHANNEL FLOW BASED UPON AVERAGE CONDITIONS IN SHALLOW ROADSIDE DITCH. FOR MORE ACCURACY USE ACTUAL QUANTITIES AND CHANNEL GEOMETRY.

FIGURE : 5 - 2

Table 5.1

Acceptable Values of Runoff Coefficient "C"

For Rational Method

Land Use	Value of C
Residential:	
Single Family (Lots < 1.0 Acre)	0.50
Single Family (Lots > 1.0 Acre)	0.40
Multi-family Detached (4 Units)	0.65
Multi-family Attached	0.75
Commercial:	
Industrial	0.75
Business	0.85
Business (downtown)	0.95
Parks & Cemeteries:	0.25
Playgrounds:	0.30
Railroad Yard Areas:	0.35
Pavements:	0.95
Roofs:	0.95
Natural Ground	0.30

Runoff Coefficient for Overland Flow Nomographs

Impervious Soils (heavy)	0.70
Impervious Soils with Turf	0.55
Slightly Pervious Soils	0.45
Slightly Pervious Soils with Turf	0.35
Moderately Pervious Soils	0.25
Moderately Pervious Soils with Turf	0.15

Table 5.2

Coefficients Adjustment Factors*

Frequency Storm	Adjustment Factor
0 – 10 yr.	1.0
25 yr.	1.1
50 yr.	1.2
100 yr.	1.25

*The product shall not give a "C" greater than 1.0.

2. Weighted Average Coefficient

A drainage area under investigation may consist of several varied drainage surface; therefore, a weighted coefficient in accordance with the respective areas would be utilized. For example, a drainage area contributing to an inlet might consist of 1 acre of asphalt paving having a coefficient of 0.95 and 3 acres of unimproved land having a coefficient of 0.3, the average coefficient would be 0.46.

B. Rainfall Intensity

For urban storm drainage, when using the Rational Method, the intensity shall be based upon the time of concentration and the selected storm frequency protection. Figures 4-1 and 4-2 shall be used for this purpose.

C. Time of Concentration

When using the Rational Method to estimate runoff from small drainage areas, the time of concentration may be determined by using the nomographs labeled as Figures 5-1 and Figure 5-2. Other generally accepted procedures for determining time of concentration may also be used if deemed appropriate for the particular circumstances by the City Engineer. In no event, however, shall times of concentration less than fifteen (15) minutes be used unless the specific situation is formally identified as unique and such is required by the City Engineer. A unique situation, wherein a time of concentration less than fifteen (15) minutes would be warranted, would involve at least a drainage area wherein the cumulative impermeability exceeds eighty percent (80%), the constructed drainage system would result in inordinately high rates of runoff, and potential flash flooding would unacceptably impair public uses.

5.3 Hydrograph Method

A. Any acceptable synthetic unit hydrograph may be used. Following is an example of an acceptable hydrograph.

B. Synthetic Unit Hydrograph

For ungaged watersheds greater than 200 acres, runoff will be determined using Synder's Synthetic Unit Graph relationship which may be referenced in the U.S. Army Corp of Engineers Engineering Manual (EM-1110-2-1405), entitled "Flood-Hydrograph Analysis and Computations," dated August 31, 1959 and Chapter 16 U.S. Army Corp of Engineer Manual (EM 1110 - 2 – 1417) entitled "Engineering and Design Flood-Runoff Analysis" dated 31 August 1994, or other applicable hydrologic literature. The following parameters have been adopted for use with the Synders Synthetic relationships: C_t=1.2; 640 C_p=500.

These values are not to be considered inflexible, but are intended as guidelines when more specific data is not available.

C. Gaged Watershed Unit Hydrograph

When feasible, unit hydrographs should be developed from the hydrologic records of a gaged watershed. Values of C_t and 640 C_p used in Snyder's Equation for Synthetic Unit Hydrograph Development for an ungaged watershed may be derived from the hydrological records of nearby gaged watersheds having similar characteristics.

D. Critical Rainfall Arrangement

Critical rainfall adjustment should be considered when utilizing the hydrograph runoff procedures. Rainfall totals should be rearranged, or critically arranged, to more correctly model actual storm patterns. Rainfall will be distributed accordingly in most hydrologic computer programs such as the HEC-1 flood hydrograph package developed by the U.S. Army Corps of Engineers. HEC-1 utilizes the procedures outlined in the U.S. Army Corps of Engineers Engineering Manual (EM-1110-2-1411), entitled, "Standard Project Flood Determination," dated March 26. 1952 (revised March 1965). This manual is recommended for reference when critically arranging rainfall by hand.

E. Initial Losses

The NRCS curve method should be used when using for hydraulic models with an initial abstraction Ia = .25 x S

$$\text{Where } S = \frac{1000}{CN} - 10$$

Where CN is the curve number for the condition.

6.0 DETENTION STORAGE

6.1 General

Detention is a means of attenuating the increased runoff associated with the development of drainage basins. Detention storage ponds should have the capacity and outlet works capable of reducing increased peak flows for each of the 2-year through 100-year frequency storms. The net flow rate from a detention storage pond shall not exceed pre-development runoff rates from the drainage basin.

Because an area used for detention of runoff may have other uses, the size, shape, and slope(s) of the detention facility should be compatible with such auxiliary uses of the facility. In addition, the allowable depth of water for the design recurrence interval and the length of time that stored water remains in the facility should also be compatible with the other uses of the facility.

6.2 Design (see Appendage A for example of Rational Method)

A. General

The following detention facility computations shall be submitted and reviewed for approval.

1. Existing pre-development runoff hydrographs for the 2-year, 5-year, 10-year, 25-year, 50-year and 100-year storms.
2. 2-year, 5-year, 10-year, 25-year, 50-year and 100-year hydrographs representing developed conditions.
3. A depth/elevation versus storage graph for the detention basin.
4. A depth/elevation versus discharge graph for the detention basin outlet works.
5. Reservoir routing computations of the developed conditions 2, 5, 10, 25 and 100-year hydrographs through the detention facility and associated outflow hydrographs.

B. Runoff Hydrographs

As discussed in Chapter 5, the runoff hydrographs for areas of 200 acres or greater will be developed using Snyder's Synthetic Unit Hydrograph procedure, or other approved methods.

For drainage areas less than 200 acres, the Rational Method, as shown in the example shown in Appendage A or the Modified Rational Method (MRM) shall be utilized to construct the runoff hydrograph.

C. Depth/Elevation – Storage Relationship

A depth/elevation storage relationship is a graph of depth/elevation of stored water vs. storage volume for the detention basin. The depth/elevation is usually placed on the ordinate, and the storage volume on the abscissa. The units for depth can be inches or feet. The units for elevation would be feet. The units for volume can be cubic feet, acre-inches, and acre-feet. The storage volume at a particular depth is always the total volume of storage below that depth.

D. Depth/Elevation – Discharge Relationship

A depth/elevation discharge relationship is expressed by a graph of depth/elevation vs. discharge rate. The depth/elevation is usually placed on the ordinate, and the discharge rate on the abscissa. The units for depth can be inches or feet. The units for elevation would be feet. The units for outflow rate can be cubic feet per second. The outflow rate at a particular depth is the summation of the outflow rates from all outlet structures which are discharging water at that particular depth.

6.3 Basin Design vs. Storm Frequency and Duration

Detention basins designed for use in the Abilene area should be designed to limit post development runoff to predevelopment rates for each of the 2-year through 100-year storms. Analysis of these storms with various durations should be carried out to ascertain that the storage space and outlet works of a detention basin are indeed capable of alternating all design storms of varying durations in accordance with Abilene standards.

6.4 Berms

Berms will not be permitted where the "toe" of the outside berm will be closer then 5 feet from the ROW line. There must be room for sidewalks near the ROW line of all streets.

6.5 Regional Detention Ponds

The City of Abilene encourage the use of larger Regional Ponds over a large number of smaller ponds in the same area's if owner or owners can work together to create one pond for general use.

7.0 CHANNELS

7.1 General

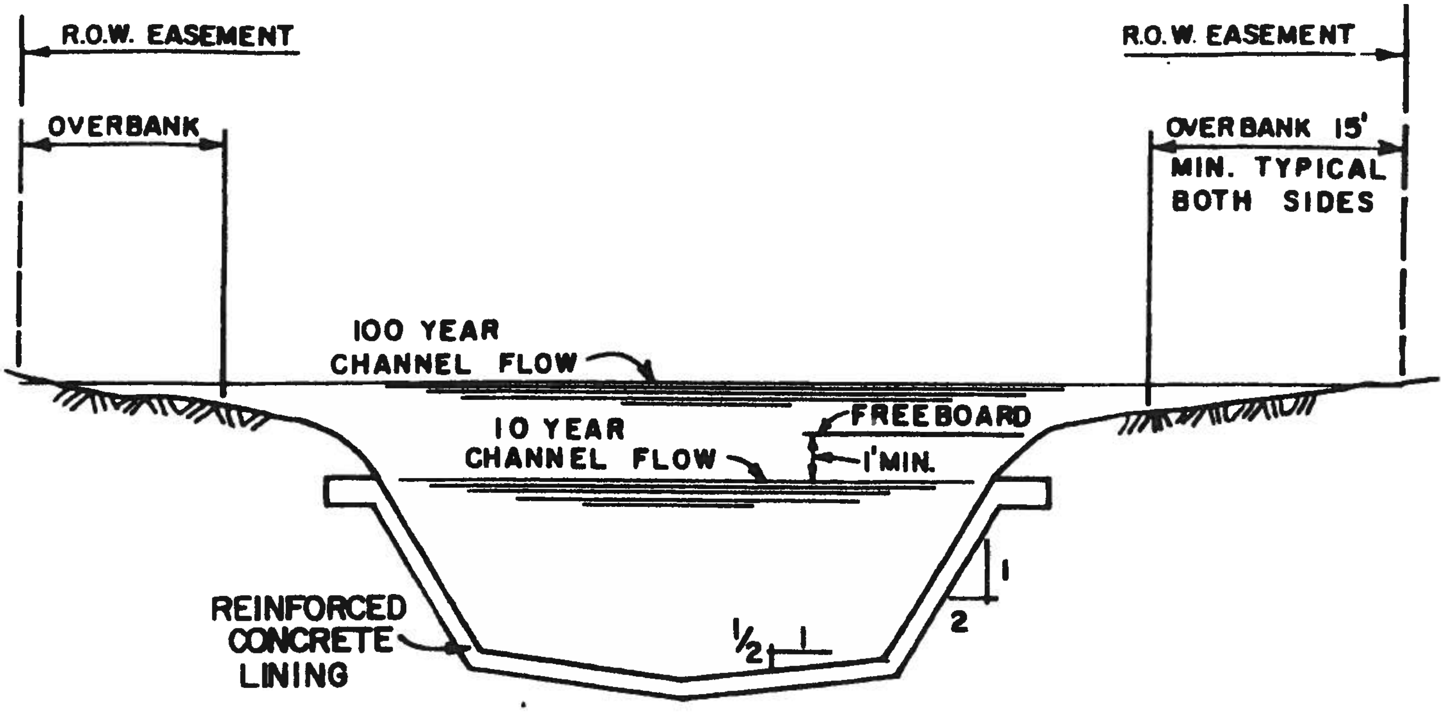
Channels exist in nature as creeks and streams and provide the natural conveyance for storm runoff. Likewise in urban storm drainage, constructed channels may improve the natural system by providing the conveyance needed for large quantities of storm runoff. Utilization of open channels for urban storm drainage, in addition to the requirements in Section 2, requires additional considerations. Channels may be advantageous because of greater capacity and normally lower cost, however, consideration must be given to the extended right-of-way requirement, safety hazards in residential areas, and maintenance costs.

Hydraulically, open channels are characterized by a concentrated flow having a free water surface. The design of open channels should provide a channel cross section of sufficient size to adequately convey the design flow of water and prevent flooding. Section 2.2 and Figure 7-1 set forth basic criteria defining utilization, capacities, shape, and lining requirements for channels.

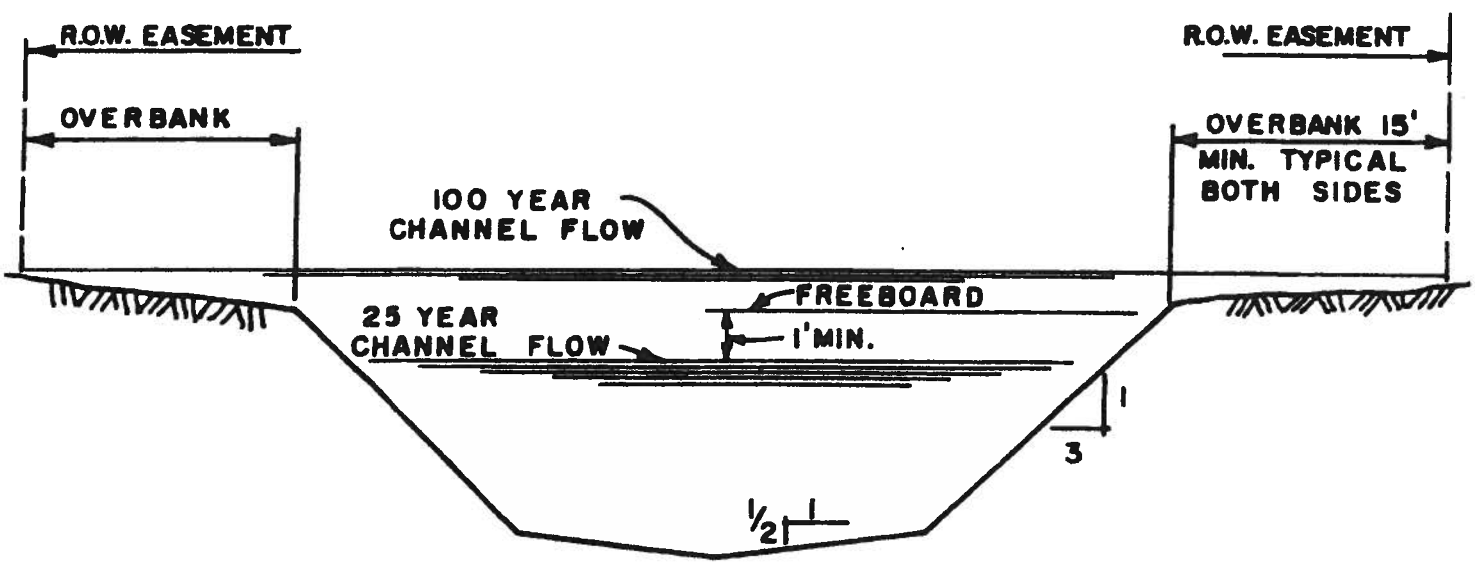
7.2 Discharge Criteria

Design flows in natural and improved channels and through bridges or culverts and other structures associated with a particular channel shall be based on the higher of the following:

1. The Abilene Flood Insurance Study.
2. Design flows frequency as calculated by design criteria given in Chapter 5.



MINIMUM CONCRETE LINED CHANNEL SECTION



MINIMUM IMPROVED CHANNEL SECTION

CHANNEL SECTIONS

FIGURE 7-1

7.3 Velocity

With higher velocities it will become necessary to provide channel lining to prevent erosion, therefore, maximum velocities of flow for various channel linings are established in Table 7.1 to give the Engineer a guide for providing channel protection. Table 7.1 is only a guide and with the advent of new technologies, linings other than those listed may be used if approved by the City Engineer. Where practical, channel stepping, flow retarding structures, or other suitable methods may be utilized to control high velocities and thereby prevent erosion.

Conversely, consideration should be given to minimum velocities and grades to prevent silting. Recommended velocities for unlined channels are as given in Table 7.1.

7.4 Freeboard

Major channels with built-up levees should be provided with 3 feet of freeboard. All other improved channels with built-up levees should be provided with 1 foot of freeboard. Consideration for additional freeboard should be given when a channel flows through an area where extensive damage would occur as a result of overflow.

7.5 Water Surface Profiles

A water surface profile must be computed for all channels and shown on all final drawings. Standard acceptable backwater methods or computer routines may be utilized. All losses due to changes in velocity, drops, bridge openings and other obstructions must be considered.

7.6 Maintaining the natural storage Volume

Maintaining the natural storage Volume provided in the floodplain will be required in channel calculations.

Table 7.1

Recommended Maximum Channel Velocities

Channel Material	Maximum Channel Velocity, fps
Fine Sand	2.5
Coarse Sand	4.0
Fine Gravel	6.0
Earth	
Sandy Silt	2.5
Silt Clay	3.5
Clay	6.0
Grass Lined Earth	
Bermuda Grass – Sandy Silt	6.0
- Silt Clay	8.0
Poor Rock (usually sedimentary)	10.0
Soft Sandstone	8.0
Soft Shale	3.5
Good Rock (usually igneous or hard metamorphic)	12.0
Reinforced Concrete Lining	15.0

8.0 FLOW IN STREETS AND STORM DRAIN INLET DESIGN

8.1 Gutter Flow

Drainage of stormwater flow in gutters is based upon the hydraulics of open channels. Permissible use of gutters for stormwater conveyance will be based upon limitations set forth in Section 2.3A.

8.2 Inlet Use

The following conditions exist for the various use of inlets.

A. Combination curb inlets should be only where space behind curb prohibits other inlet types.

B. Grate inlets should be used only where space restrictions prohibit other inlets or at locations with no curb.

C. No depressed inlets shall be used on arterials and freeways unless they are clearly outside all traffic lanes.

D. Inlets shall be placed preferably at the upstream side of street intersections, at low points, or where the gutter flow exceeds its limits. Inlets shall be located on off streets or alleys when possible.

E. Water flowing in gutters of arterials and expressways should be picked up prior to super-elevated sections to prevent water flowing across the street for up to a 5-year storm as per Section 2.3A4.

F. In super-elevated sections of divided arterials, inlets placed against the center medians shall have no gutter depression and shall intercept gutter flow at the point of vertical curvature.

8.3 Inlet Types

Inlets must be utilized at the point at which street flow conditions begin to approach or exceed the storm runoff limitations set forth in Section 2.3A. The final selection of inlet types will be based upon hydraulic performance, safety requirements and economics.

Inlets in sumps, curb inlets on grade, depressed curb inlets on grade, recessed curb inlets and combinations thereof are acceptable means of intercepting excess surface runoff.

The following guidelines shall be used in the design of inlets to be located on streets.

A. Maximum permissible depression for depressed curb inlets (depression measured from gutter line) in residential and collector streets shall be 5 inches.

B. Maximum permissible depression for depressed curb inlets (depression measured from gutter line) in divided and undivided arterials and freeways shall be 2 ½" unless specifically approved by the City Engineer.

C. Inlets shall have a 6" minimum throat opening.

D. Recessed inlets shall not interfere with the intended use of a sidewalk.

E. Inlets should be designed and located with pedestrian and bicycle traffic in mind.

F. The following reductions in calculated inlet capacities shall be utilized to account for blockage of the inlets by trash and debris:

Inlet Type	Capacity Reduction
5" depressed curb inlet	10 percent
Drop inlets in sumps	10 percent
Grate inlets	25 percent
Combination grate & curb inlet in a sump	20 percent

G. The capacity of a combination curb and grate inlet on grade shall be considered to be 90 percent of the sum of their individual capacities (allowing for reduction due to clogging). This will also apply to a combination depressed curb and grate inlet on grade.

H. The capacity of a combination curb and grate inlet in a sump shall be considered the sum of their individual capacities (allowing for clogging).

9.0 CULVERTS

9.1 General

Culverts are an integral part of any storm drainage system. The function of a drainage culvert is to pass stormwater flow from the upstream side of an embankment to the downstream side without creating excessive downstream velocities, submerging embankment or causing excessive backwater.

9.2 Quantity of Flow

The quantity of design flow shall be determined in accordance with Chapter 5 of this manual.

9.3 Headwalls and Endwalls

Headwalls will be either straight parallel headwalls, flared headwalls, paved sloped entrances, warped headwalls or pre-formed headwalls, or as approved by the City Engineer with or without aprons depending on site conditions. The following guidelines are suggested:

Conditions	Headwall and Endwall Type
Approach velocities below 6 fps. Approach channel undefined. Formation of backwater pool's acceptable. No downstream channel protection required.	Straight parallel
Approach velocities between 6-10 Approach channel well defined.	Flared (wings of flared fps. walls located with respect To axis of the approach channel velocity)
Approach velocities 8 fps or greater. Approach channel well defined.	Warped (suggested for use only for large drainage Installations with limited right-of-way)

9.4 Culvert Hydraulics

The hydraulic design of culverts will be based upon design aids found in the Texas Highway Department specifications and standard plans, or other suitable material.

9.5 Discharge Velocities

The velocity of discharge from culverts should be limited as shown in Table 9.1. Consideration must be given to the effect of high velocities and turbulence on the channel, adjoining property and embankment.

Table 9.1

Culvert Discharge Velocities

Culvert Discharging on to	Maximum allowable Velocity (fps)
Earth (sandy)	6
Earth (calcareous clay)	8
Earth (sodded, vegetated)	8
Hard Shale	10
Rock or concrete	15

See Appendage B for additional maximum Velocities values.

9.6 Erosion Control

In certain instances, rip rap protection and energy dissipaters downstream of culverts may prove to be more economical solutions to high outlet velocities than resizing of a culvert. Engineering judgment will dictate the design of energy dissipaters and sizing and placing of rip rap. The Technical Report. No. FHWA/RO-82/011, "Scour at Culvert Outlets in Mixed Bed Materials" U.S. Department of Transportation dated September 1982, is one source of information available regarding the effects of scour downstream of culvert outlets.

9.7 Culvert Type

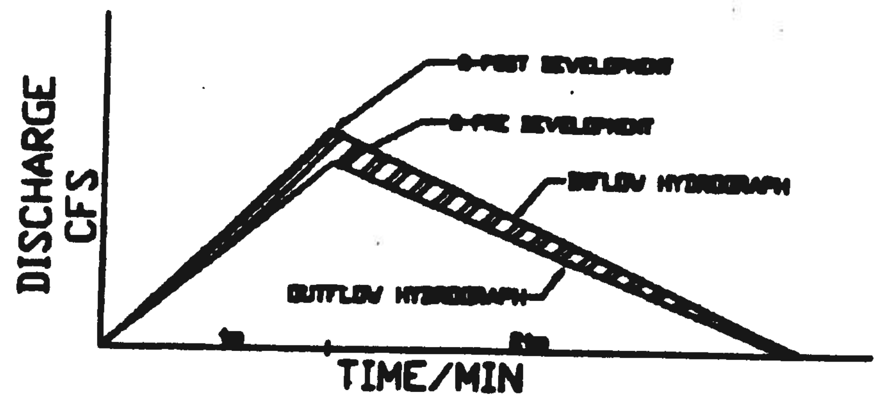
The selection of the type of culvert utilized and its shape is left open to individual judgment based on local conditions.

Appendage A

Detention Design

CITY OF ABILENE

By: Terry Pribble, P.E.
January 2000



Detention storage equals difference between the volume of the inflow hydrograph and the outflow hydrograph in the case where tcj (tp) is the same as with the minimum tc as in the Abilene drainage standards.

The volume of the hydrograph is equal the area under the hydrograph connected for time, the area under ONE hydrograph equals.

$$V = A = 1/2(TP)(Qp) \times 60 + 1/2(2tp)(Qp) \times 60$$

$$V = 30tpQp + 60tpQp = 90tpQp$$

$$S = 90tp(Qpost - Qpre)$$

Add 15% to compensate for hydraulic routing.

$$St = 90tp(Qpost - Qpre) \times 1.15$$

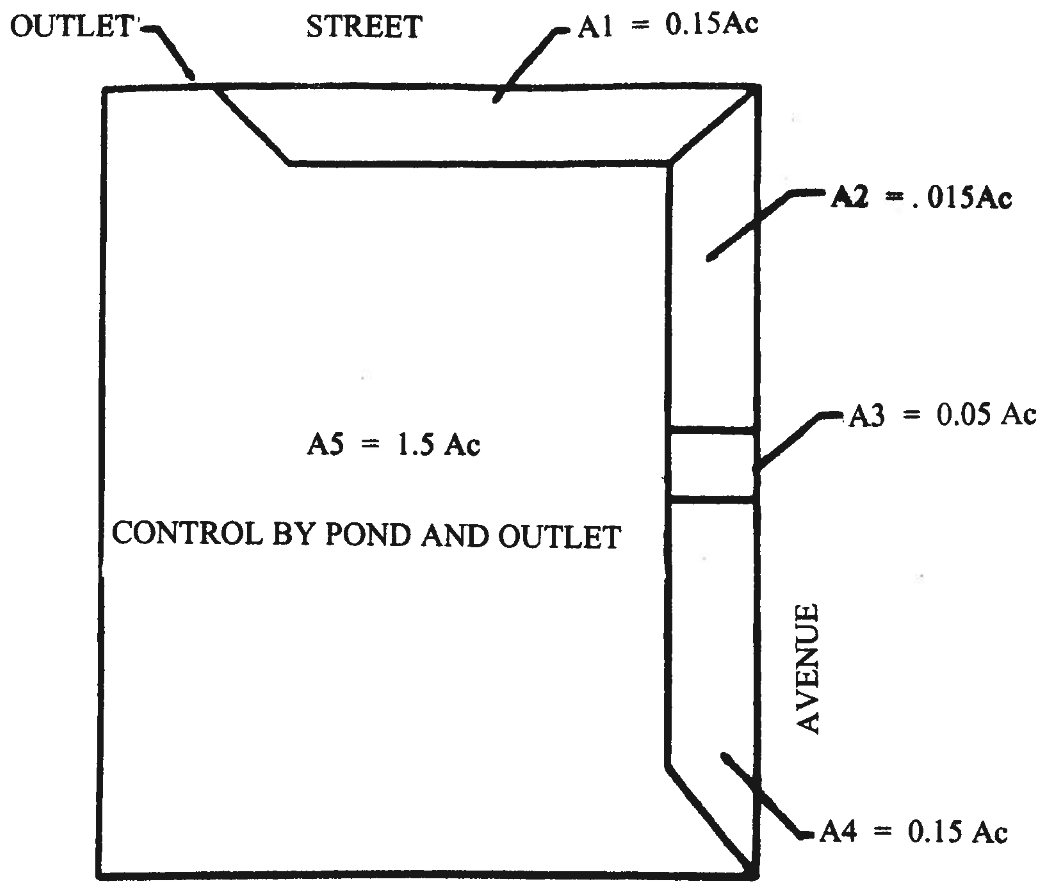
V: Volume

A: Area

Tp: Time to Peak in minutes

S: Storage in cubic feet

St: Total Storage in cubic feet



Given Total Area = 2.0 Acres

Predevelopment C = 0.35 Page 5.2, old value in Abilene Drainage Standards prior to May 1, 2007

Post development C for Areas A_1, A_2, and A_4 = 0.35

Post development C for Areas A_3, and A_5 = 0.95

$A_1 = 0.15Ac$

$A_2 = 0.15Ac$

$A_3 = 0.05Ac$

$A_4 = 0.15Ac$

$A_5 = 1.5Ac$

$A_T = 2.00Ac$

Tc = 15 min pre and post development

Page 5.3, Abilene Drainage Standards

Adjustment factor $F_5 = 1.0$

$F_{100} = 1.25$

Page 5.3 Abilene Drainage Standards

$I_{100} = 7.15$

$I_5 = 4.75$ Figure IV-1, Abilene Drainage Standards

$Q = CFIA$
NOTE: If CF>1.0, use 1.0

EXISTING CONDITION
$Q_{5pre} = 0.35 \times 1.0 \times 2.0 \times 4.75 = 3.325$ cfs
$Q_{100pre} = 0.35 \times 1.25 \times 2.0 \times 7.15 = 6.256$ cfs

DEVELOPED CONDITIONS
Area of (C @ 0.35) = $A_1 + A_2 + A_4 = 0.15 + 0.15 + 0.15 = 0.45$ Acres
Area of (C @ 0.95) = $A_3 + A_5 = 0.05 + 1.5 = 1.55$ Acres

$$C \text{ weighted} = \frac{(0.45 \times 0.35) + (1.55 \times 0.95)}{2.0} = \frac{1.63}{2.0} = 0.815$$

$Q_{5Dev} = 0.815 \times 1 \times 2.0 \times 4.75 = 7.743$ cfs
*
$Q_{100Dev} = 1.0 \times 2 \times 7.15 \times = 14.30$ cfs

*0.815 x 1.25 = 1.0188 > 1.0 use 1.0

DETENTION REQUIREMENT

Based on 100 year	$S = 90\ tc\ (Q_{Dev} - Q_{pre})$	tc = 15 min
	$S = 90 \times 15\ (14.3 - 6.256) =$	10859.4 ft^3
	Add 15% for hydraulic routing	1628.91 ft^3
		12488.31 ft^3 minimum detention volume required

VOLUME PROVIDED
Based on $V = 1/3\ (b) \times (A + B + (AB)^{1/2})$
Where b = height between contours in feet
Where A and B = surface area of contours in square feet

This example assumed:

Elevation	Area (ft^2)
1760.0	0.0
1760.5	508.0
1761.5	15608.0
1762.0	19807.0

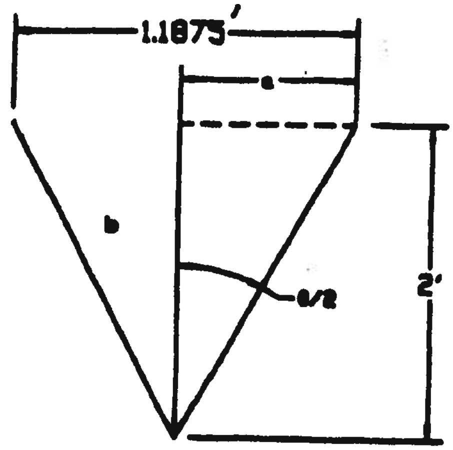
b = 0.5 between 1760.0 and 1760.5
$V = 1/3 \times 0.5 \times (508 + 0 + (0 \times 508)^{1/2} = 84.67\ ft^3$

Sum of Volume
84.67 ft
($6395.28\ ft^3$)

b = 1.0 between 1760.5 and 1761.5
$V = 1/3 \times 1 \times (508 + 15608)^{1/2}) = 6310.61\ ft^3$
b = 0.5 between 1761.5 and 1762.0
$V = 1/3 \times 0.5 \times (15608 + 19807 + (15608 \times 19807)^{1/2}) = 8832.93\ ft^2$ 15228.21 ft^3
15228.21 > 12488.21 ok

Constructed notch to the below requirement, use 1' 2 ¼ = 1.1875'

$$\tan\frac{\theta}{2} = \frac{\;\frac{1.1875}{2}\;}{2} = 0.296875$$



$$\frac{\theta}{2} = \tan^{-1}(0.296875)$$

$$\frac{\theta}{2} = 16.534838$$

$$\theta = 33.069676^\circ \qquad 33^\circ\, 4'\, 10.8''$$

Check outflow from weir and runoff directed to streets

$$Q_w = 8/15 \times 0.58 \times (2g)^{1/2} \times \tan\frac{\theta}{2} \times H^{5/2}$$

$$Q_w = 8/15 \times 0.58 \times (64.4)^{1/2} \times \tan\frac{\theta}{2} \times 2^{5/2}$$

$$Q_w = 8/15 \times 0.58 \times (64.4)^{1/2} \times 0.296875 \times 2^{5/2}$$

$$Q_w = 4.169 \text{ cfs}$$

a = 1.1875/2
b = 2
tan = 8/2 = a/b = 1.1875/2/2

Total 100 year developed runoff

$$Q_{100} = Q_w + Q_{TI\ street} = 4.169 + 1.408 + 0.3575 = 5.9345 \text{ cfs}$$

Page 3 Page 3

5.9345 < 6.256 ok

Page 2 Existing 100 year discharge

Check water balance for 5 year
Determine storage volume for 5 year storm

$$V = 90tc(Q_{Dev\,5} - Q_{pre\,5}) = 90 \times 15(7.743 - 3.325) = 5964.3 \text{ ft}^3$$

tc = 15 min Page 3 Page 2

$d_5 = 1.45$ for Page 7

$$Q_w = 8/15 \times 0.58 \times (2g)^{1/2} \times \tan\frac{\theta}{2} H^{5/2}$$

$$Q_w = 8/15 \times 0.58 \times (64.4)^{1/2} \times 0.296875 \times 1.45^{5/2}$$

$$Q_w = 1.866 \text{ cfs}$$

Total 5 year runoff $= Q_w = Q_{street}$
$= 1.866 + 0.748 + 0.2256 = 2.8396$ cfs

Page 3 Page 3

2.8396 cfs < 3.325 cfs

Page 2 Existing 5 year

DESIGN ALTERNATE 1 SIZE OUTLET FOR 100 YEAR DISCHARGE AND CHECK 5 YEAR

Discharge from Area A_1, A_2, and A_4 developed

$A_1 = 0.15$ Ac
$A_2 = 0.15$ Ac
$A_4 = \underline{0.15 \text{ Ac}}$
0.45 Ac

$Q_5 = 0.35 \times 1.0 \times 0.45 \times 4.75 = 0.748$ cfs
$Q_{100} = 0.35 \times 1.25 \times 0.45 \times 7.15 = 1.408$ cfs

Discharge from Area A_3, developed

$A_3 = 0.05$ AC

$Q_{5\,Dev} = 0.95 \times 1.0 \times 0.05 \times 4.75 = 0.2256$

use 1.0

$Q_{100\,Dev} = (0.95 \times 1.25) \times 0.05 \times 7.15 = 0.3575$ cfs

Allowable outflow = Q = 6.256 – 1.408 – 0.3575 = 4.4905 cfs*

Used V-notch weir where $Q = \frac{8}{15} \times 0.58 \times (2g)^{1/2} \times \tan\frac{\theta}{2} \times H^{5/2}$

*from Page 2 Pre Dev 100 year

H = 1762
$\underline{1760}$
2.0 see volume calculations

$$4.4905 = \frac{8}{15} \times 0.58 \times (64.4)^{1/2} \times \tan\frac{\theta}{2} \times 2^{5/2}$$

$$\tan\frac{\theta}{2} = \frac{4.4905}{14.0425} = 0.3197791$$

$$\frac{\theta}{2} = \tan^{-1}(0.3197791)$$

$$\frac{\theta}{2} = 17.733189^\circ$$

$$\theta = 35.466378^\circ \text{or } 35^\circ\ 27'\ 59''$$

Top width of notch

$2 \times 2 \times 0.3197791 = 1.2791$ feet

DESIGN ALTERNATE 2 SIZE OUTLET FOR 5 YEAR

$A_1 = 0.15Ac$ $\quad Q_5 = 0.35 \times 1 \times 0.45 \times 4.75 = 0.748$ cfs

$A_2 = \underline{0.15Ac}$ $\quad Q_{100} = 0.35 \times 1.25 \times 0.45 \times 7.15 = 1.408$ cfs

$A_4 = 0.45Ac$ $\quad$ Discharge from Areas A_1, A_2, A_4 Develop Condition

$A_3 = 0.05Ac$ $\quad$ Discharge form Are A_3

$Q_{5Dev} = 0.95 \times 1.0 \times 0.05 \times 4.75 = 0.2256$ cfs

use 1.0

$Q_{100Dev} = 0.95 \times 1.25 \times 0.05 \times 7.15 = 0.3575$ cfs

Outlet = 3.325 – 1.408 – 0.3575 = 1.5595 cfs

Page 2 $\quad$ Dev_{100} $\quad$ Dev_{100} $\quad$ use 1.56 cfs

Pre Dev_5

Use V-notch weir

$$Q = 8/15 \times 0.58 \times (2g)^{1/2} \times \tan \frac{\theta}{2} \times H^{5/2}$$

H = 2.0' $\quad$ Page 3

$$1.5595 = 8/15 \times 0.58 \times (64.4)^{1/2} \times \tan \frac{\theta}{2} \times 2^{5/2}$$

Use 1.56 cfs

$$\tan \frac{\theta}{2} = \frac{1.56}{14.0425} = 0.110913$$

$$\tan \frac{\theta}{2} = \tan^{-1}(0.110913)$$

$$\frac{\theta}{2} = 6.3390722^\circ$$

$$\theta = 12.678144^\circ$$

Top width = 2 x 2 x 0.110913 = 0.443652

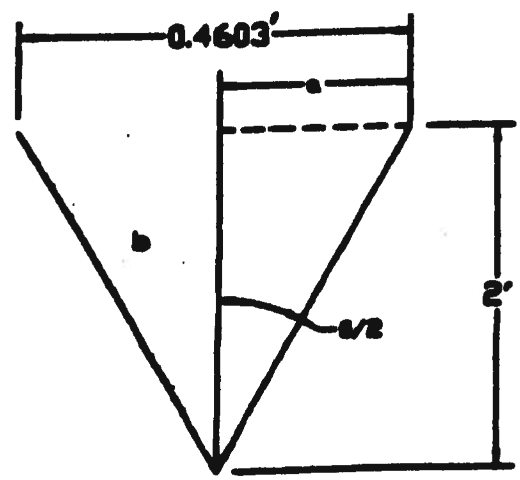
use 4 7/8" = 0.4063

$$\tan \frac{\theta}{2} = \frac{\underline{0.4063}}{\frac{2}{2}} = 0.101575$$

$$\frac{\theta}{2} = \tan^{-1}(0.101575)$$

$$\frac{\theta}{2} = 5.7999265^\circ$$

$\theta = 11.599853^\circ$ or $11^\circ 36'$



a = 0.4063/2

b = 2

tan 8/2 = a/b = 0.4063/2/2

Determine flow through water

$$Q_w = 8/15 \times 0.58 \times \tan \frac{0}{2} \times (64.4)^{1/2} \times H^{5/2}$$

$$Q_w = 8/15 \times 0.58 \times (0.101575) \times (64.4)^{1/2} \times 2^{5/2}$$

$$Q_w = 1.4264 \text{ cfs}$$

Total outflow 100 year = $Q_{weir} + Q_{tcstreet}$ (Page 5)

= 1.4264 + 1.408 + 0.3575 = 3.1919 cfs

3.1919 cfs < 6.256 cfs

Pre 100 year

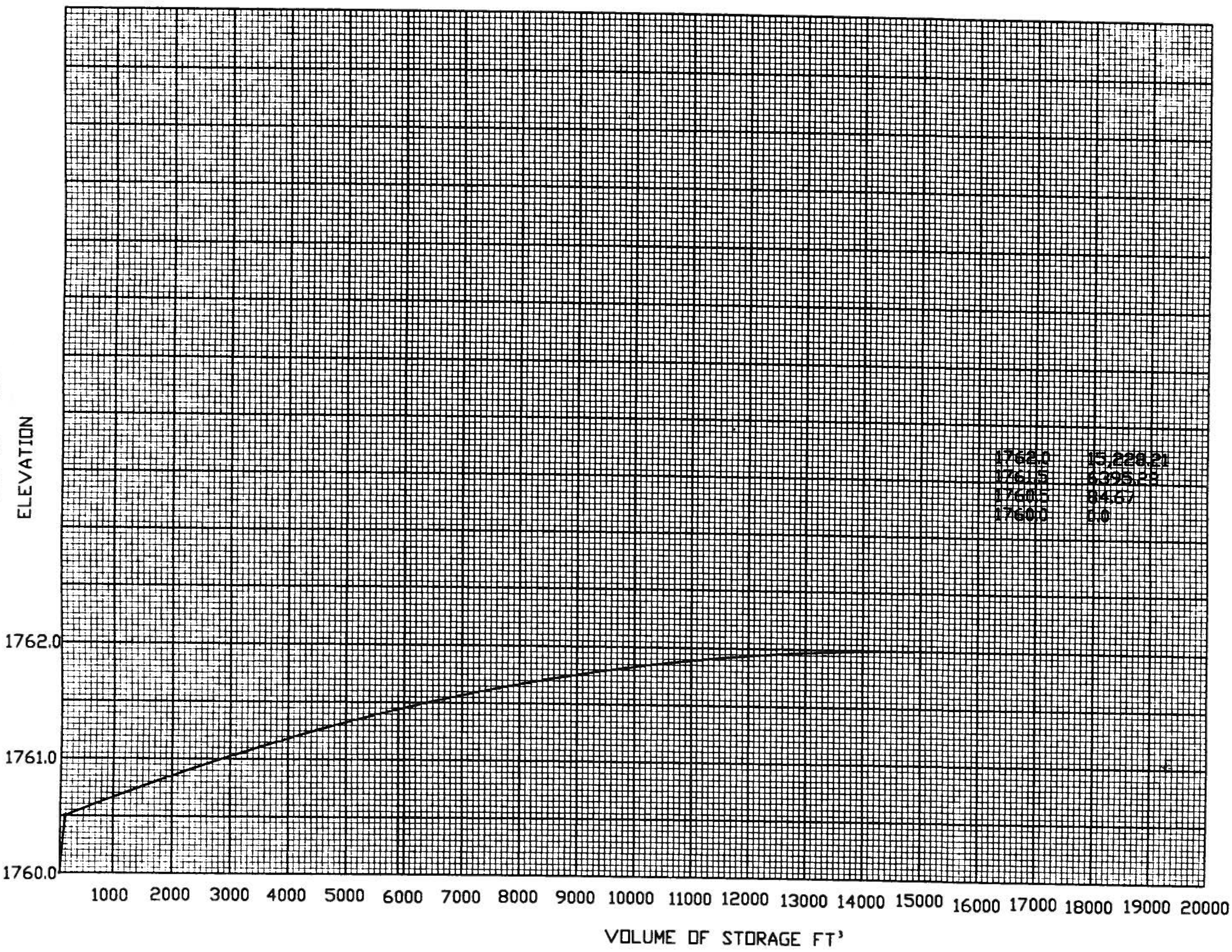
Page 2

Total outflow 5 year = $Q_{weir} + Q_{tcstreet}$ (Page 5)

= 1.4264 + 0.748 + 0.2256 = 2.40 cfs

2.40 cfs < 3.325 cfs

Q_{5pre} Page 2


1762.0 15,228.21
1761.5 6,395.28
1760.5 84.67
1760.0 0.0
ELEVATION
1762.0
1761.0
1760.0
1000 2000 3000 4000 5000 6000 7000 8000 9000 10000 11000 12000 13000 14000 15000 16000 17000 18000 19000 20000
VOLUME OF STORAGE FT³

EXAMPLE HEC 1 ROUTING AS THE EXAMPLE SHOWN DRAINAGE STANDARDS	DETENTION POND IN APPENDAGE "A" OF CITY OF ABILENE					
*****************************	**********************	***********	************	*************	***********	***********
Pre-development		ID	A	C	C*A	
			Acres			
		1	2	0.3	0.6	
		2				
		A total	2		0.6	
			Cw		0.3	
Storm freq.	Cw	Fc	Cw*fc	Cw*fc used	I	Q
years					inch/hour	cfs
2	0.3	1	0.3	0.3	3.9	2.340
5	0.3	1	0.3	0.3	4.75	2.850
10	0.3	1	0.3	0.3	5.25	3.150
25	0.3	1.1	0.33	0.33	6	3.960
50	0.3	1.2	0.36	0.36	6.6	4.752
100	0.3	1.25	0.375	0.375	7.15	5.363
**************************	**************************	***********	*************	*************	************	***********
Post development		ID	A	C	C*A	
		1	0.15	0.3	0.045	
		2	0.15	0.3	0.045	
		3	0.5	0.95	0.048	
		4	0.15	0.3	0.045	
		5	1.5	0.95	1.425	
		A total	2		1.608	
			Cw		0.804	
Storm freq.	Cw	Fc	Cw*fc	Cw*fc used	I	Q
years					inch/hour	cfs
2	0.80375	1	0.80375	0.80375	3.9	6.269
5	0.80375	1	0.80375	0.80375	4.75	7.636
10	0.80375	1	0.80375	0.80375	5.25	8.439
25	0.80375	1.1	0.884125	0.884125	6	10.610
50	0.80375	1.2	0.9645	0.9645	6.6	12.731
100	0.80375	1.25	1	1	7.15	14.300

To Page 2

To Page 2

Storm freq.		Post Del		Pre Del	delt Q	Volume				
years		Q		Q	cfs	ft^3				
		cfs		cfs						
5	From Page 1	7.636	From Page 1	2.850	4.786	6460.594				
10		8.439		3.150	5.289	7140.656				
25		10.610		3.960	6.650	8976.825				
50		12.731		4.752	7.979	10772.190		FT^3		
100		14.300		5.363	8.938	12065.625	1.15	13875.469	0.319	acre-feet
********************		*************		*************	**************	************	************	************	************	

Uncontrol flow	ID	A	C	C*A
	3	0.05	0.95	0.0475
	A total	0.05		0.0475
		Cw		0.95

Storm freq.	Cw	Fc	Cw*fc	Cw*fc used	I	Q
years					inch/hour	cfs
2	0.95	1	0.95	0.95	3.9	0.107
5	0.95	1	0.95	0.95	4.75	0.131
10	0.95	1	0.95	0.95	5.25	0.144
25	0.95	1.1	1.045	1	6	0.182
50	0.95	1.2	1.14	1	6.6	0.218
100	0.95	1.25	1.1875	1	7.15	0.246
************************	*****************	****************	*************	**************	************	************

To Page 3

FLOWS TO POND

ID	A	C	C*A
1	0.15	0.3	0.045
2	0.15	0.3	0.045
4	0.15	0.3	0.045
5	1.5	0.95	1.425
A total	1.95		1.56
	Cw		0.8

Storm freq. years	Cw	fc	Cw*fc	Cw*fc used	I inch/hour	Q cfs	
2	0.8	1	0.8	0.8	3.9	6.08	To Page 4
5	0.8	1	0.8	0.8	4.75	7.41	
10	0.8	1	0.8	0.8	5.25	8.19	
25	0.8	1.1	0.88	0.88	6	10.30	
50	0.8	1.2	0.96	0.96	6.6	12.36	
100	0.8	1.25	1	1	7.15	13.94	

****

STORAGE & DISCHARGE PROVIDED

V = 1*3*b*(A+B+(A*B)^0.5

weir equation

Q = (8/15*.58* tan (0/2) * ((64.4)^0.5 * d ^ 2.5

Tan (0/2) = W/2/H

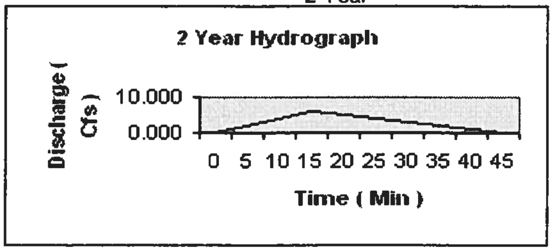
H Height of weir feet	W Top width of weir feet	tan (0/2)	Number of weirs
2	1.1875	0.29688	1

Elevation feet	area of contour FT^2	depth between C feet	Volume At elevation ft^3	Total volume at elevation ft^3	Total volume at elevation acre-feet		Depth of flow feet		Discharge of weirs cfs		Elevation feet
1760.00	0	0	0	0	0	To Page 5	0	To Page 5	0	To Page 5	1760.00
1760.50	508	0.50	84.66666667	84.66666667	0.001943679		0.50		0.130277		1760.50
1761.50	15608	1.00	6310.607953	6395.274619	0.146815303		1.50		2.03082		1761.50
1762.00	19807	0.50	8832.932543	15228.20716	0.349591533		2.00		4.168869		1762.00

TOP OF BERM 1762.00

Storm Frequency years	Pre-dev Q cfs		uncontrol Q cfs		Rounted HEC 1 flow cfs		TOTAL OUTFLOW Q cfs		ROUNTED HEC 1 STAGE FEET	
								ok		
2	2.34	From Page 1	0.19	From Page 2	1.78	From Page 6	1.97	ok	1761.37	From Page 5 & 6
5	2.85		0.23		2.13		2.36	ok	1761.52	
10	3.15		0.25		2.31		2.56	ok	1761.57	
25	3.96		0.30		2.81		3.11	ok	1761.68	
50	4.75		0.33		3.31		3.64	ok	1761.80	
100	5.36		0.36		3.69		4.05	ok	1761.89	

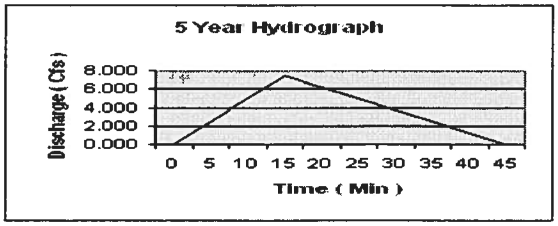
2 Year



From Page 3

0	0	6.080	0.000
5	0.333333	6.080	2.027
10	0.666667	6.080	4.053
15	1	6.080	6.080
20	0.833333	6.080	5.067
25	0.666667	6.080	4.053
30	0.5	6.080	3.040
35	0.33333	6.080	2.027
40	0.166667	6.080	1.013
45	0	6.080	0.000

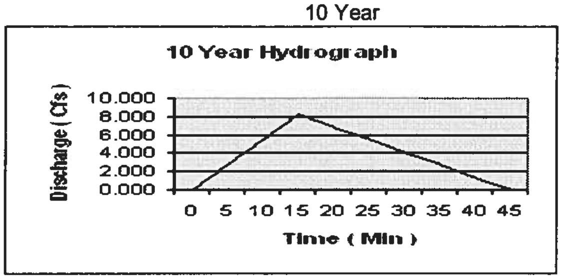
5 Year



From Page 3

0	0	7.410	0.000
5	0.333333	7.410	2.470
10	0.666667	7.410	4.940
15	1	7.410	7.410
20	0.833333	7.410	6.175
25	0.666667	7.410	4.940
30	0.5	7.410	3.705
35	0.33333	7.410	2.470
40	0.166667	7.410	1.235
45	0	7.410	0.000

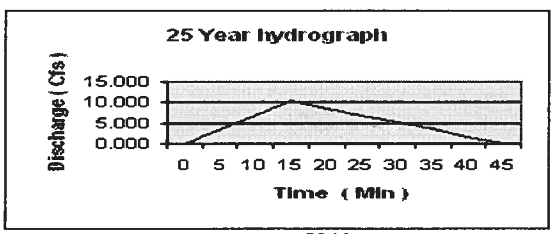
10 Year



From Page 3

0	0	8.190	0.000
5	0.333333	8.190	2.730
10	0.666667	8.190	5.460
15	1	8.190	8.190
20	0.833333	8.190	6.825
25	0.666667	8.190	5.460
30	0.5	8.190	4.095
35	0.33333	8.190	2.730
40	0.166667	8.190	1.365
45	0	8.190	0.000

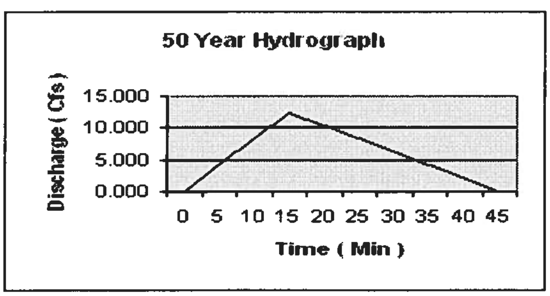
25 Year



From Page 3

0	0	10.300	0.000
5	0.333333	10.300	3.433
10	0.666667	10.300	6.867
15	1	10.300	10.300
20	0.833333	10.300	8.583
25	0.666667	10.300	6.867
30	0.5	10.300	5.150
35	0.33333	10.300	3.433
40	0.166667	10.300	1.717
45	0	10.300	0.000

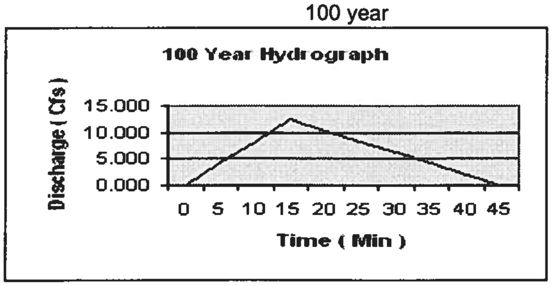
50 Year



From Page 3

0	0	12.360	0.000
5	0.333333	12.360	4.120
10	0.666667	12.360	8.240
15	1	12.360	12.360
20	0.833333	12.360	10.300
25	0.666667	12.360	8.240
30	0.5	12.360	6.180
35	0.33333	12.360	4.120
40	0.166667	12.360	2.060
45	0	12.360	0.000

100 year



From Page 3

0	0	13.940	0.000
5	0.333333	13.940	4.647
10	0.666667	13.940	9.293
15	1	13.940	13.940
20	0.833333	13.940	11.617
25	0.666667	13.940	9.293
30	0.5	13.940	6.970
35	0.33333	13.940	4.647
40	0.166667	13.940	2.323
45	0	13.940	0.000

HEC-1 INPUT

LINE												
1	ID	EXAMPLE HEC 1 ROUNTING DETENTION POND										
2	ID	AS THE EXAMPLE SHOWN IN APPENDAGE A DETENTION DESIGN										
3	ID	OF THE DRAINAGE STANDARD OF THE CITY OF ABIELNE										
4	ID	1 - V-NOTCH WEIR 2.0 FEET IN HEIGHT AND 1.1875 FEET IN WIDTH AT TOP										
5	IT	1	17JUL9	0000	145							
6	JP	6										
7	VS	Pond	Pond	Pond	Pond	Pond	Pond					
8	VV	2.11	2.21	2.31	2.41	2.51	2.61					
9	KK	Lot										
	*2 YEAR EVENT											
10	IN	5	17JUL9	0000								
11	QI	0.000	2.027	4.053	6.080	5.067	4.053	3.040	2.027	1.013	0.000	From Page 4
12	KP	2										
	* 5 YEAR EVENT											
13	IN	5	17JUL9	0000								
14	QI	0.000	2.470	4.940	7.410	6.175	4.940	3.705	2.470	1.235	0.000	From Page 4
	*10 YEAR EVENT											
15	KP	3										
16	IN	5	17JUL9	0000								
17	QI	0.000	2.730	5.460	8.190	6.825	5.460	4.095	2.730	1.365	0.000	From Page 4
	*25 YEAR EVENT											
18	KP	4										
19	IN	5	17JUL9	0000								
20	QI	0.000	3.433	6.8767	10.300	8.583	6.867	5.150	3.433	1.717	0.000	From Page 4
	*50 YEAR EVENT											
21	KP	5										
22	IN	5	17JUL9	0000								
23	QI	0.000	4.1200	8.240	12.360	10.300	8.240	6.180	4.120	2.060	0.000	From Page 4

HEC-1 INPUT

```
LINE     ID.........1.........2.........3.........4.........5.........6.........7.........8.........9.......Page

 24      KP        6
 25      IN        5    17JUL9    0.000
 26      QI    0.000     4.647    9.293   13.940   11.617    9.293    6.970    4.647    2.323    0.000   From Page 4

 27      KK     Pond
 28      RS        1      ELEV   1760.0
 29      SV    0.000     0.002    0.147    0.350
 30      SQ    0.000     0.130     2.03     4.17   From Page 3
 31      SE   1760.0    1760.5   1761.5   1762.0
 32      ZZ
                  * PEAK STAGES IN FEET*
                         1            STAGE   1761.3
                                      TIME      0.60
                         2            STAGE   1761.5
                                      TIME      0.60
                         3            STAGE   1761.5
                                      TIME      0.60   To Page 3
                         4            STAGE   1761.6
                                      TIME      0.62
                         5            STAGE   1761.8
                                      TIME      0.62
                         6            STAGE   1761.8
                                      TIME      0.62

        MAX     1.78      2.13     2.31     2.81     3.31     3.69
        MIN      .00       .00      .00      .00      .00      .00   To Page 3
        AVE     1.27      1.53     1.69     2.11     2.49     2.78
***NORMAL END OF HEC-1***
```

Appendage B

Supplemental Technical Guide

CITY OF ABILENE

By: Terry Pribble, P.E.
March 2002

Curve Number (CN)
Based on City of Abilene Zoning and Hydrologic Soil Groups

Land Use Category	Square Feet	% Impervious	Curve Number for Hydrologic Soil Groups A	B	C	D
Pasture or Range Poor Condition			68	79	86	89
Zoning						
RS6	6000	50	83	89	92	94
RS8	8000	45	82	88	91	93
RS12	12000	40	80	87	91	93
AO	87000	25	76	84	89	91
Office		85	94	95	96	97
Limited Commercial		85	94	95	96	97
Shopping Center		95	97	97	97	98
General Commercial		85	94	95	96	97
Center Business		85	94	95	96	97
Heavy Commercial		100	98	98	98	98
RM1		64	87	91	94	95
RM2		39	80	86	91	93
RM3		40	80	87	91	93
Patio Homes		50	83	93	96	97
Town Houses		50	83	93	96	97

Equation CNc = CNp + ((Pimp/100)*(98-CNp)
CN with connected connected impervious areas TR55 Second Edition Appendix F

Rational C Values for Slopes 0% - 2%
Based on City of Abilene Zoning and Hydrologic Soil Groups

Zoning	Square Feet	% Impervious	Curve Number for Hydrologic Soil Groups A	B	C	D
RS6	6000.00	50	0.52	0.54	0.57	0.59
RS8	8000.00	45	0.48	0.50	0.53	0.56
RS12	12000.00	40	0.44	0.46	0.49	0.52
AO	87000.00	25	0.31	0.33	0.37	0.40
Office		85	0.82	0.83	0.84	0.84
Limited Commercial		85	0.82	0.83	0.84	0.84
Shopping Center		95	0.95	0.95	0.95	0.95
General Commercial		85	0.82	0.83	0.84	0.84
Center Business		85	0.82	0.83	0.84	0.84
Heavy Commercial		100	0.95	0.95	0.95	0.95
RM 1		64	0.64	0.65	0.68	0.69
RM 2		39	0.43	0.45	0.48	0.51
RM 3		40	0.44	0.46	0.49	0.52
Patio homes		50	0.52	0.54	0.57	0.59
Town Houses		50	0.52	0.54	0.57	0.59

Based on the MSD Louisville and Jefferson County Metropolian Sewer District
Drainage Manual dated January 1, 2001.

Raitonal C Values for Slopes 2% - 7%
Based on City of Abilene Zoning and Hydrologic Soil Groups

Zoning	Square Feet	% Impervious	Curve Number for Hydrologic Soil Groups A	B	C	D
RS6	6000.00	50	0.55	0.57	0.59	0.61
RS8	8000.00	45	0.51	0.53	0.55	0.58
RS12	12000.00	40	0.47	0.49	0.52	0.54
AO	87000.00	25	0.35	0.38	0.41	0.44
Office		85	0.83	0.84	0.85	0.85
Limited Commercial		85	0.83	0.84	0.85	0.85
Shopping Center		95	0.95	0.95	0.95	0.95
General Commercial		85	0.83	0.84	0.85	0.85
Center Business		85	0.83	0.84	0.85	0.85
Heavy Commercial		100	0.95	0.95	0.95	0.95
RM 1		64	0.66	0.67	0.69	0.71
RM 2		39	0.46	0.48	0.51	0.53
RM 3		40	0.47	0.49	0.52	0.54
Patio homes		50	0.55	0.57	0.59	0.61
Town Houses		50	0.55	0.57	0.59	0.61

Based on the MSD Lousiville and Jefferson County Metropolian Sewer District Drainage Manual dated January 1, 2001.

Raitonal C Values for Slopes 7%+
Based on City of Abilene Zoning and Hydrologic Soil Groups

Zoning	Square Feet	A	B	C	D
			Curve Number for Hydrologic Soil Groups		
RS6	6000.00	0.58	0.61	0.64	0.66
RS8	8000.00	0.44	0.47	0.60	0.63
RS12	12000.00	0.51	0.54	0.57	0.60
AO	87000.00	0.39	0.43	0.48	0.52
Office		0.84	0.85	0.86	0.86
Limited Commercial		0.84	0.85	0.86	0.86
Shopping Center		0.95	0.95	0.95	0.95
General Commercial		0.84	0.85	0.86	0.86
Center Business		0.84	0.85	0.86	0.86
Heavy Commercial		0.95	0.95	0.95	0.95
RM 1		0.61	0.63	0.66	0.68
RM 2		0.50	0.53	0.57	0.60
RM 3		0.51	0.54	0.57	0.60
Patio homes		0.58	0.61	0.64	0.66
Town Houses		0.58	0.61	0.64	0.66

Based on the MSD Lousiville and Jefferson County Metropolian Sewer District Drainage Manual dated January 1, 2001.

Table A2
Typical Roughness Coefficients for Open Channels
(Reference: Chow, Ven Te, 1959; Open-Channel Hydraulics)

Type of Channel and Description	Minimum	Normal	Maximum
EXCAVATED OR DREDGED			
a. Earth, straight and uniform			
1. Clean, recently constructed	0.016	0.018	0.020
2. Clean, after weathering	0.018	0.022	0.025
3. Gravel, uniform section, clean	0.022	0.025	0.030
4. With short glass, few weeds	0.022	0.027	0.033
b. Earth, winding and sluggish			
1. No vegetation	0.023	0.025	0.030
2. Grass, some weeds	0.025	0.030	0.033
3. Dense weeds or aquatic plants in deep channels	0.030	0.035	0.040
4. Earth bottom and rubble sides	0.028	0.030	0.035
5. Stony bottom and weedy banks	0.025	0.035	0.040
6. Cobble bottom and clean sides	0.030	0.040	0.050
c. Dragline-excavated or dredged			
1. No vegetation	0.025	0.028	0.033
2. Light brush on banks	0.035	0.050	0.060
d. Rock cuts			
1. Smooth and uniform	0.025	0.035	0.040
2. Jagged and irregular	0.035	0.040	0.050
e. Channels not maintained, weeds and brush uncut			
1. Dense weeds, high as flow depth	0.050	0.080	0.120
2. Clean bottom, brush on sides	0.040	0.050	0.080
3. Same, highest stage of flow	0.045	0.070	0.110
4. Dense brush, high stage	0.080	0.100	0.140

Table A2 (continued)
Typical Roughness Coefficients for Open Channels
(Reference: Chow, Ven Te, 1959; Open-Channel Hydraulics)

Type of Channel and Description	Minimum	Normal	Maximum
NATURAL STREAMS			
Minor streams (top width at flood stage < 100 feet)			
a. Streams on plain			
1. Clean, straight, full stage, no rifts or deep pools	0.25	0.030	0.033
2. Same as above, but more stones and weeds	0.030	0.035	0.040
3. Clean, winding, some pools and shoals	0.033	0.040	0.045
4. Same as above, but more weeds and stones	0.035	0.045	0.050
5. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
6. Same as 4, but more stones	0.045	0.050	0.060
7. Sluggish reaches, weedy deep pools 0.050	0.070	0.080	
8. Very weedy reaches, deep pools, or floodways with Heavy stand of timber and underbrush 0.075	0.100	0.150	
LINED OR BUILT-UP CHANNELS			
a. Corrugated Metal	0.021	0.025	0.030
b. Concrete			
1. Trowel finish	0.011	0.013	0.015
2. Float finish	0.013	0.015	0.016
3. Finished, with gravel on bottom	0.015	0.017	0.020
4. Unfinished 0.014	0.017	0.020	
5. Gunite, good section	0.016	0.019	0.023
6. Gunite, wavy section	0.018	0.022	0.025
7. On good excavated rock	0.017	0.020	
8. On irregular excavated rock	0.022	0.027	

Table A2 (continued)
Typical Roughness Coefficients for Open Channels
(Reference: Chow, Ven Te, 1959; Open-Channel Hydraulics)

Type of Channel and Description	Minimum	Normal	Maximum
c. Concrete bottom, float finished with sides of			
1. Dressed stone in mortar	0.015	0.017	0.020
2. Random stone in mortar	0.017	0.020	0.024
3. Cement rubble masonry, plastered	0.016	0.020	0.024
4. Cement rubble masonry	0.020	0.025	0.030
5. Dry rubble or riprap	0.020	0.030	0.035
d. Gravel bottom with sides of			
1. Formed concrete	0.017	0.020	0.025
2. Random stone in mortar	0.020	0.023	0.026
3. Dry rubble or riprap	0.023	0.033	0.036
e. Asphalt			
1. Smooth	0.013	0.013	
2. Rough	0.016	0.016	
f. Vegetal lining	0.030		0.500

Table A3
Manning's Roughness Coefficients for Straight Channels
Without Shrubbery or Trees

	Depth of Flow of 0.7 to 1.5 feet	Depth of Flow greater Than 3.0 feet
Bermuda grass, Buffalo grass, Kentucky Bluegrass		
a. Mowed to 2 inches	0.035	0.030
b. Length 4-6 inches	0.040	0.030
Good stand, any grass		
a. Length of 12 inches	0.070	0.035
b. Length of 24 inches	0.100	0.035
Fair stand, any grass		
a. Length of 12 inches	0.060	0.035
b. Length of 24 inches	0.070	0.035

Table 10-4

MAXIMUM PERMISSIBLE VELOCITIES FOR EARTH CHANNELS WITH VARIED GRASS LININGS AND SLOPES

Channel Slope	Lining	Permissible Mean Channel Velocity * (ft/sec)
0 – 5%	Sodded grass	7
	Bermudagrass	6
	Reed canarygrass	5
	Tall fence	5
	Kentucky bluegrass	5
	Grass-legume mixture	4
	Red fescue	2.5
	Redtop	2.5
	Sericea lespedeza	2.5
	Annual lespedeza	2.5
	Small grains (temporary)	2.5
5 – 10%	Sodded grass	6
	Bermudagrass	5
	Reed canarygrass	4
	Tall fence	4
	Kentucky bluegrass	4
	Grass-legume mixture	3
Greater than 10%	Sodded grass	5
	Bermudagrass	4
	Reed canarygrass	3
	Tall fence	3
	Kentucky bluegrass	3

*For highly erodible soils, decrease permissible velocities by 25%.

*Grass lined channel are dependent upon assurances of continuous growth and maintenance of grass.

FROM REVISION TO CITY OF COLORADO SPRINGS/EL PASO COUNTY DRAINAGE CRITERIA MANULA 10-12-94

HYPOTHETICAL STORM DATA IN INCHES
FOR ABILENE, TX

Frequency Percent	Years	0.0833	0.25	1	2	3	6	12	24
50	2	0.47	0.93	1.65	1.93	2.10	2.49	2.95	3.50
20	5	0.56	1.13	2.13	2.51	2.80	3.40	4.20	4.75
10	10	0.62	1.28	2.47	3.10	3.35	4.10	4.80	5.50
4	25	0.71	1.49	2.94	3.52	4.10	4.80	5.60	6.60
2	50	0.79	1.66	3.31	4.10	4.60	5.30	6.30	7.50
1	100	0.86	1.82	3.67	4.70	5.10	6.15	7.20	8.50
0.2	500	1.08	2.28	4.12	5.80	6.40	7.70	8.90	10.70

Sources Used

NRCS EFN2 Software
USCE

"DRAFT FEASIBILITY REPORT & ENVIRONMENT ASSESSMENT LOCAL FLOOD PROTECTION ELM CREEK WATERSHEDABILENE, TEXAS" VOLUME II APPENDICES DATED 1990 RAINFALL CHARTS FROM "HANDS-ON HEC1" BY DODSON

Appendage C

Detention Pond Location Guidelines

CITY OF ABILENE

By: Bob Lindley, P.E.
June 2007

DETENTION POND LOCATION GUIDELINES
And
Permit Form

CITY OF ABILENE

It is not of best interest to anyone to have drainage detention ponds located in the back yard of a home. If the detention pond is to be dedicated to the City of Abilene, it shall be located in other areas of the property and not in an area fenced into the family dwelling. If there are no alternative locations for a detention pond the detention pond shall be included in an "open easement". Specifically the "open easement" will not permit a fence or other structures inside the area.

Existing Detention Ponds that were constructed prior to these "Guidelines", and the owner desires a fence, the fence must be designed by a Registered Engineer. The Engineer shall make the determination that proposed slots in the fence will adequately drain the water from the pond and not interfere with the designed outflow. The slots shall be located in the lowest elevation of the pond.

The property owner shall also provide a way for the City Employees to access the property for maintenance work. This will include access of equipment as well as employees. If access is not provided for equipment, the City Employees will remove section of fence for access. The City will not be responsible for damage to the fence or in the yard.

By signing this form the owner is agreeing to this method of action and the City will consider this as a permit for the owner to build a fence and for the City Employees to enter identified private property and perform maintenance work.

Owner: ______________________________ Date:_______________
Address: ______________________________
Phone: ______________________________

Inspector: ______________________________ Date:_______________

City
Engineer: ______________________________ Date:_________________

Appendage D

Guidelines
Flume Opening in Streets

CITY OF ABILENE

By: Bob Lindley, P.E.
June 2007

Guidelines
Flume Opening in Streets

The throat width of outlet flumes of streets shall be calculated using the following equation L = Q100 / 3.0*H^ (3/2) where L equals the Length in feet of the opening into the street, Q100 = the storm discharge in the street at the inlet during the 100 year storm event, H equals the difference in feet between the invert elevation of the inlet and the lowest natural grade in the Parkway. The flume at the property line shall be designed using the manning equation for the 100 year storm event. For abnormal and/or unusual conditions the City Engineer may consider adjustments.